BORED PILE RETAINING WALL SOLUTIONS FOR EARTHQUAKE SLIP 6 AT OHAU POINT, KAIKOURA, NEW ZEALAND

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ABSTRACT: The 14 November 2016 M_w 7.8 Kaik ura Earthquake triggered a number of landslips along the Kaik ura coastline. Ohau Point is located approximately 26 km north of Kaik ura and was the site of massive landslide which destroyed the road and rail transport corridors. This paper describes the bored pile retaining wall solutions that were developed as part of the North Canterbury Transport Infrastructure Recovery (NCTIR) earthquake recovery program of works for the New Zealand Transport Agency (NZTA) and KiwiRail. The subject site is named as NCTIR Site 6 and the subject bored pile retaining walls are named RTW 3.

Keywords: Bored pile retaining wall, Kaikoura earthquake and bored pile construction

1. INTRODUCTION

The M_w 7.8 Kaik ura Earthquake occurred on 14 November 2016 at 12.02 am. The earthquake initiated along a complete network of several existing faults with the rupture trending towards north for a distance of approximately 170km. The faults which were ruptured during the 2016 Kaikoura earthquake included the Humps, Conway-Chartwelll, Upper Kowhai, Hundalee, Fidget, Jordan, Kekerengu, Papatea, Hope and Needles faults.

Figure 1 shows the location of the faults which are currently known to have ruptured during the 2016 Kaikoura earthquake.

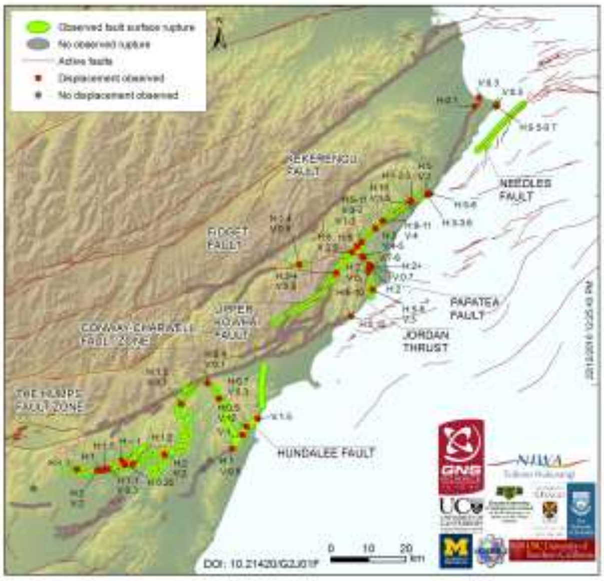

Figure 1-2016 Kaikoura Earthquake Fault rupture map.

The Kaikoura earthquake caused over NZD $ 1.2 billion of damage to the public transport infrastructure along Inland route 70, State Highway (SH 1) and main north line Railway (MNL). This damage was primarily due to major landslides and slips above and/or below the transport corridors.

The MNL and SH 1 were closed between Clarence and Kaik ura SH 1. This is the main transport link between the Picton and Christchurch and the only rail link. The alternative transport link is via state highways 63, 6, 65 and 7 (alternate route). This route is an alpine route and subjected to closure over winter months. The route is also not designed to accommodate large volumes of heavy traffic. A number of small communities were isolated and the closures had significant impact on the regional economy.

The North Canterbury Transport Infrastructure Recovery (NCTIR) is an alliance partnership between Fulton Hogan, Downer, Higgins HEB Construction and the New Zealand Government. The New Zealand Transport Agency and KiwiRail are board members and funders for the programme and represent the New Zealand Government on the NCTIR project. The objective of the NCTIR program of works is to restore the State highway and rail transport networks which were damaged by the Kaik ura earthquake and create a more resilient infrastructure along Inland route 70, State Highway (SH1) and the Main North Line (MNL) railway.

The major landslide which was located at the south side of Ohau Point was named as Slip 6 by NCTIR. The Slip 6 landslide material had a total volume of approximately 110,000 m^3. Figure 2 shows the Slip 6 landslide a few days after the Kaik ura earthquake.

Figure 2-The Site 6 landslide a few days after the 14 November 2016 Earthquake

At Site 6 the NZTA carriageway is located on the seaward side and the KiwiRail railway is located on the landward side of the repair corridor. Figure 3 shows the location of NCTIR Site 6.

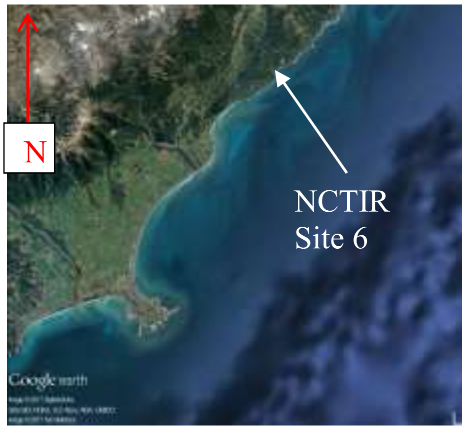

Figure 3-Satellite image showing the location of NCTIR Site 6 - Image from Google Earth Pro ©

As part of the proposed repair works in the slip 6 area the road is to be widened and a new geogrid reinforced soil and mass block seawall constructed. New road and rail alignment is proposed for a safe transport corridor. The new road and rail realignment at Site 6 also requires a new bored pile retaining system to be constructed between the new road and rail alignments in order to retain the ground above the road and support the loads from rail infrastructure, rail operations and a new rock fall protection bund. The proposed wall enabled a reduction in height of an average of 3m over more than 100m. The retained height of this bored pile wall which is named RTW 3, typically varies between 2.0m and 5.0m high. RTW 3 has been subdivided into eight sections and named RTW 3 A1 to RTW 3 D3. This subdivision was primarily made due to reasons of construction progress and sequence. Figure 6 and 7, which are towards the end of this paper, present typical cross-sections through the Slip 6 repair works

Figure 4 below provides an oblique drone photograph of the Site 6- RTW 3 location, which was taken during late 2017. The new Site 6 seawall (under construction), old rail and road alignments are clearly visible in Photograph 4.

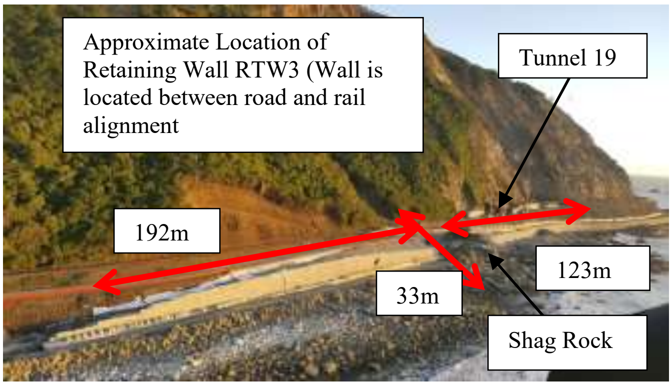

Figure 4-New Site 6 seawall & RTW 3 Bored Pile Retaining Wall (under construction)

It is worth noting that several new structures and mitigation have been implemented at Site 6 including: a rock fall bund, debris flow structures, rock fall mitigation catchment fences, sluicing of the landslide debris and blasting of the rocks. These works were designed by the NCTIR ‘Slope Team’ and were substantially completed before construction of the new structures and RTW3 commenced, to mitigate the risk of landslide and rockfall during construction.

2. SITE CROSS-SECTIONS

The Site 6 RTW 3 is divided into eight sub-sections, based on the site geology, construction program and /or various retaining heights and different loading cases on top of the walls.

Figures 6 and 7 present typical present cross sections through the transport corridors, seawalls and RTW3 at Site 6

3. DESIGN CRITERIA AND PHILOSPHY

NZTA Bridge Manual V3 (BM3) Table 2.2 has been adopted as the design basis for the RTW 3 retaining structures. In accordance with the recommendations outlined in BM3 Table 2.2, the proposed Site 6 RTW 3 works have been designed as an importance level 3 (1/2500 year earthquake event) structure as they are to support a primary lifeline route (i.e. the MNL).

, the RTW 3 retaining walls were designed to meet the deflection and settlement criteria requirements of the relevant KiwiRail specifications and BM3. Table 1 below summarises the target wall deflection, movement and settlement criteria which was adopted during the design of Site 6 RTW 3.

Table 1: Summary of Geotechnical Target Design Criteria

Parameter	**Limit**
Track Settlement (static)	20mm
Track Settlement (Seismic)	Tracks to be checked & re-levelled after any significant seismic event
Maximum Wall Deflection (ULS Seismic)	100mm
Minimum Slope Stability (FOS)-Static	1.50
Minimum Slope Stability (FOS)- Seismic	1.10

3.1 Design Assumptions / Constraints

The following special issues and constraints were duly considered and accommodated in the design for Site 6 RTW 3:

1. Construction programme, Kaik ura and small communities to the north had been isolated since the Kaikoura earthquake severely impacting the wider region. The New Zealand Government and the affected community wanted ‘full operation’ to be reinstated as quickly as possible.
2. Limited space was available between the proposed new rail and road alignment. This was one of the primary drivers for the selection of a bored pile solution.
3. The seaward side of Shag Rock is a nesting area for shags and the proposed retaining structure and road alignment needed to be built around the existing rock outcrop and have minimum to nil impact on this culturally significant topographic feature.
4. Settlement and deformation of the rail infrastructure behind the retaining structure (Rail track and rock fall bund) had to be maintained below acceptable limits as published by KiwiRail.

3.2 Design Life

In accordance with NZTA and KiwiRail requirements a 100 year design life was adopted for the design of Site 6 RTW 3.

4. LOADING CRITERIA

4.1 Static and Live Loads

4.1.1 Rail Loading and Rockfall Protection Structure (Tunnel Extension)

A rail live load of 90kPa was adopted for the design of this project in accordance with the recommendations which are published in the KiwiRail Specification. A load of 100kPa has also been applied, where appropriate, to account for the weight of the rockfall protection structure (tunnel extension) which is to be constructed at the southern approach to Tunnel 19. This allowance was selected in collaboration with the designers of the tunnel extension structure.

4.1.2 Live Vehicle Loading

A live load of 30kPa has been adopted, where appropriate, during design of the retaining walls, to account for heavy construction traffic and equipment.

4.1.3 Rock Fall Protection Bund and Attenuation Energy Load

The following load allowances have been made for the rock fall protection bund and rock fall energy attenuation loads. These loads were provided by the designers of the Rock fall bund.

- Rock fall bund dead load = 55kPa.
- Rock fall energy attenuation energy load = 80kPa to 340kPa.

4.2 Seismic Loads

Immediately following the M_w Kaik ura Earthquake, the damage at Site 6 and to the existing road and rail infrastructure was observed to be predominantly due to landslip debris impact that originated in sandstone/mudstone materials above these assets. The peak horizontal ground acceleration at the site during the M_w 7.8 earthquake of 14 November 2016 is inferred from data published by the University of Canterbury as being in the order of 0.50g. As such, the site is considered to have been recently tested to between a 1 in 500 and 1 in 1000 year return period seismic event.

In terms of AS/NZS1170.5 the site is assessed to be a Class B – Rock site as it meets the following criteria:

The subgrade material comprises rock with a compressive strength of between 1 and 50 MPa, and, an average shear wave velocity greater than 360 m/s. Further, this site is underlain by materials having a compressive strength greater than 0.8 MPa and shear wave velocity greater than 300 m/s as per Class B Rock site.

The design peak horizontal seismic loading (PGA_H) value has therefore been estimated using the following equation:

$$PGA_H = C_{0\,1000} x \frac{R_u}{1.3} . f\ (g) \quad \text{(BM3 section 6.2.2)} \qquad (1)$$

Where:

f = 1.00 for a Class B site,

$C_{0\,1000}$ = 0.55 for the Kaik ura area, and,
Ru (1/2500) = 1.8

Using the above equation and assumptions, the following ULS seismic design values for the peak horizontal -acceleration were calculated:

$$PGA\ (1/2500)_H = 0.55\ x\ \frac{1.8}{1.3}\ x\ 1.00 = 0.76g$$

In general accordance with the wider project design philosophy (based on records and codes), a vertical seismic acceleration (PGA_V) value of 0.31g (approximately 40% of the horizontal PGA_H value) was used in the pseudo-static models in conjunction with the horizontal acceleration.

The design value for the vertical acceleration is in general accordance with the recommendation published in Eurocode 8.

5. GEOLOGY AND GROUND MODEL

5.1 Ground Conditions

The road and rail corridor is located between the South Pacific Ocean and the foothills of the Kaik ura mountain range. The following sub-sections provide a general description of the geological, hydro – geological and geotechnical conditions which are inferred to underlie the subject site.

5.2 Geological Setting

At the location of Site 6 the major geological unit is Pahau Terrane (Ktp). This unit is described in the relevant published geological map as well bedded and poorly bedded sandstone. These units are of early cretaceous age and are approximately 145 million years old. The site-specific geotechnical investigation results indicate a thin layer of beach gravels and/or colluvium overlies the sandstone bedrock beneath most locations along the proposed Site 6 RTW 3 footprint.

5.3 Ground and Surface Water Conditions

Fifteen machine drilled borehole investigations have been undertaken within the general area around Site 6. The bedrock level beneath the Site 6 retaining walls varied between 11 m and 14 m below the existing ground surface level (bgl) and around 1m bgl in the Shag Rock area (refer to Figure 4 for approximate location). The material overlying the bedrock is generally described as colluvium comprising medium dense silty sandy gravel and occasionally Silt with some gravel and boulders. The ground water level was generally observed to be sitting on the colluvium rock interface and at a depth of between 11 and 14m bgl.

5.3 Geotechnical Parameters

The geotechnical design parameters which were adopted during the analysis of RTW 3 are summarised in Table 2 below.

Table 2: RTW 3 Geotechnical Design Parameters

Unit Name	Soil Model	Unit Weight (kg/m³)	Cohesion (kPa)	Tension (kPa)	Friction Angle (°)	Young's Modulus (MPa)
Bedrock	Modified Hoek-Brown	2548	GSI = 30, mi = 15, σ_{ci} = 50MPa			300
Colluvium	Mohr-Coulomb	1800	7	0	35	50
Concrete	Mohr-Coulomb	2550	535	1795	35	23500

The colluvium is described in the borehole logs as a mix of silt, sand, gravel and boulder material and has uncorrected SPT N values of between 4 and 50+. As such, this material was considered to be, on average, a medium dense soil with some cohesion.

These parameters were also verified by the back-analysis of select slopes.

Table 3: Summary of the Site 6 RTW 3 configuration

Retaining Wall ID	Maximum Retaining Height	Approx Wall Length	Proposed load or Structure above the final wall	Proposed retaining system
RTW 3A1	3.0m	43.0m	New rail alignment	0.8m diameter pile at 1.8m C/C spacing , 12.0m long
RTW 3A2	2.0m	41.0m	New rail alignment	0.8m diameter pile at 2.0m C/C spacing , 9.0m long
RTW 3B (rock)	2.0m	20.0m	New rail alignment	0.8m diameter pile at 2.0m C/C spacing , 7.0m long
RTW 3B(colluvium)		68.0m		0.8m diameter pile at 2.0m C/C spacing , 9.0m long
RTW 3C	4.0m	123.0m	New rail alignment/ proposed extension of tunnel/ rock fall bund	0.9m diameter pile at 1.5m C/C spacing , 13.0m long
RTW 3D1	5.0m	20.0m	New rail alignment	0.8m diameter pile at 0.85m C/C spacing, 12.0m long, with one row of Anchor at 1.7m spacing (32mm Anchor, 200kN pre stress Load)
RTW 3D2	5.0m	26.0m	Existing sloping ground	0.8m diameter pile at 0.85m C/C spacing, 8.0m to 12.0m long with Base Slab
RTW 3D3	5.0m	7.0m	Existing sloping ground	0.8m diameter pile at 0.85m C/C spacing, 8.0m to 12.0m long with Base Slab

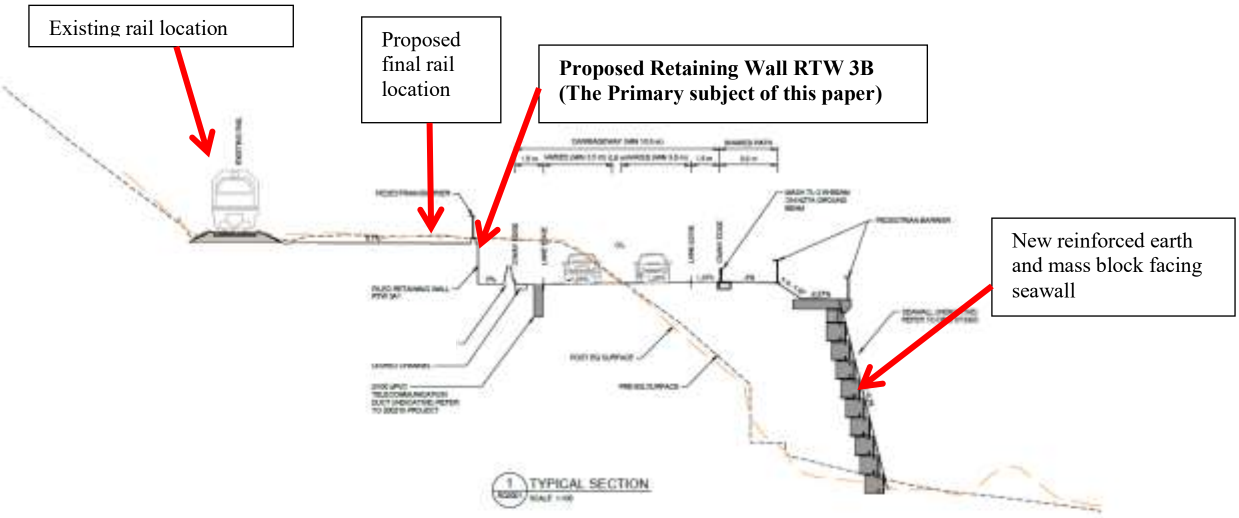

Figure 6: Typical Cross section through Site 6 RTW 3B

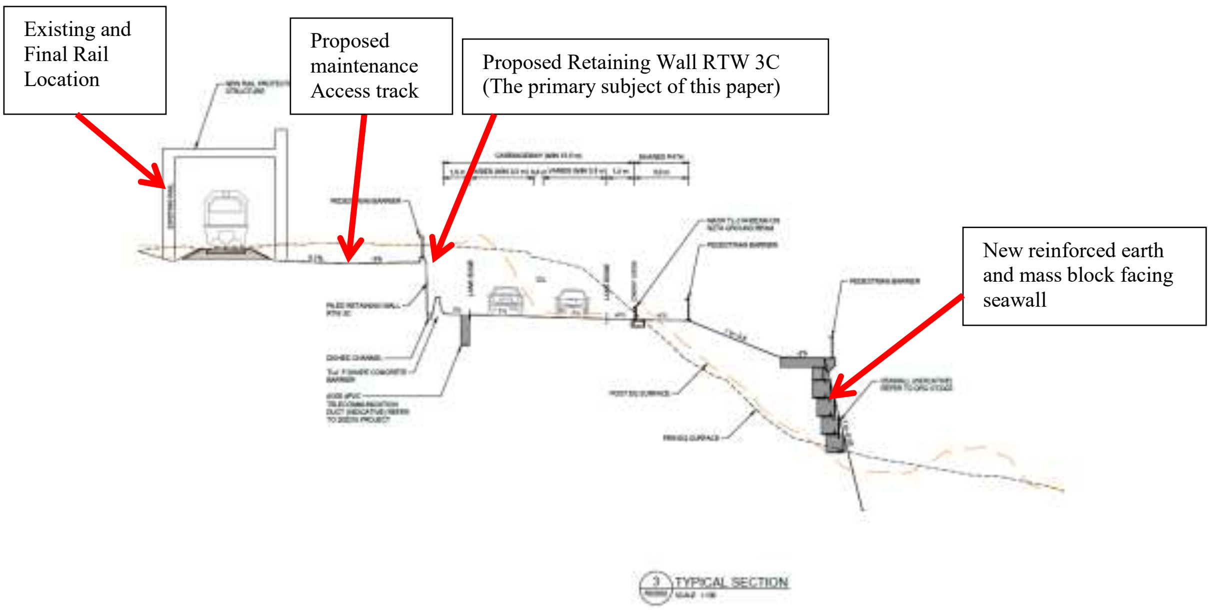

Figure 7: Typical cross section through Site 6 RTW 3C

6. DESGN OPTIONS

6.1 Assessment of Options

The preliminary options for the Site 6 RTW 3 were assessed and the preferred solution selected based on due consideration of the following key criteria:

- The likely performance of the retaining wall.
- The constructability of the retaining walls within the timeframes stipulated by the New Zealand Government to open the road and rail transportation corridors.
- Consideration of the temporary live operation of the rail on the old alignment and,
- Cost.

The following retaining wall options were considered during the options assessment phase of the Site 6 RTW 3 design.

Option 1 - No retaining walls and provide stable batters. This option was found to be infeasible due to the unavailability of space between the new road and rail alignment.

Option 2 - Cantilever pole retaining wall designs comprising steel or timber poles. These wall designs were found to be unsuitable for reasons of durability, strength and/or deflection.

Option 3 - Gabion basket retaining wall. As with Option 2, preliminary analysis indicated that this solution was unlikely to achieve a satisfactory level of performance with respect to deflection, in particular as insufficient space was available to provide geogrid 'tie backs' and enable the fastest possible reinstatement of this critical lifeline route (i.e. similar issues to options 4 below).

Option 4 - Reinforced earth wall (soil nail or MSE). This option was assessed to have significant constructability issues as it required a significant excavation to be made behind the wall and the placement of geogrids and engineered fill which would have required the relocation or removal of the rail line and ballast. This in turn had the potential to cause significant disruption to the rail service and delay reconnecting of this critical lifeline route.

Option 5 - Secant bored piles (SBP). This option was assessed to be infeasible due to construction constraints such as available cutters for secondary piles and duration.

Option 6 - Contiguous flight auger piles (CFA). The primary advantage of CFA piles was the method does not require temporary casing to support the pile holes. However, this option was assessed to be infeasible due to constructability issues in the colluvium and rock materials.

Option 7 – Conventional bored pile wall with shotcrete infill panels. This was selected as the preferred option as it was expected to meet all of the performance criteria and have the highest level of constructability.

6.2 Selection of Preferred Option

The conventional bored pile retaining wall option (Option 7 above) was identified as the preferred option primarily because it was judged to have the lowest level of construction risk, could be constructed within the available corridor width, and could achieve most if not all of the design requirements in particular rapid reconnection of this lifeline route post- earthquake. All of the other options were judged to have some technical fatal flaw.

A reinforced concrete bored pile wall with capping beam was identified from the options assessment process as the best available solution to retain the ground adjacent to the proposed road. The ground between the piles was designed to activate an arching mechanism. Megaflow drainage and concrete shotcrete facing was also provided between the piles to provide long-term durability, control ground water pressures, and, ensure an appropriate level of in-service performance was obtained. For aesthetic reasons an architectural facing is proposed to dress the exposed section of the wall, however, this architectural facing was not finalised at the time of writing this paper. Due allowance for an architectural dressing was made during the detailed design of the wall super structure.

7. GEOTECHNICAL DESIGN METHODOLOGY

The geotechnical stability and structural demands on the proposed retaining walls were analysed using the specialized software package FLAC version 8.00.448 and Mohr-Coulomb soil models. Figure 8 provides an example typical output from the FLAC Model, showing total wall displacements.

The structural demands on wall RTW 3 which were obtained from the FLAC analysis were cross checked using the geotechnical software package Wallap V6.06. For the purposes of the Wallap assessment, the cantilever walls were assessed to act as a stiff system and the anchored wall were assessed to act as a rigid system

Table 3 above summarises the key aspects of the final design which was developed for each of the eight sub sections of RTW 3.

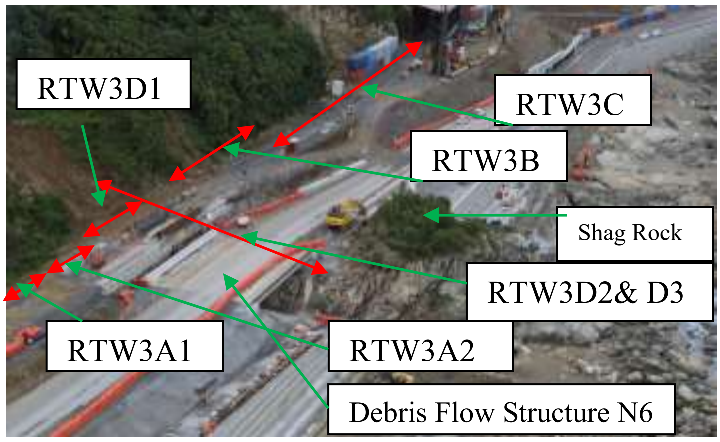

Figure 5: Aerial Photograph of complete portion of RTW 3 which is located adjacent to Shag Rock and Debris flow Structure N6

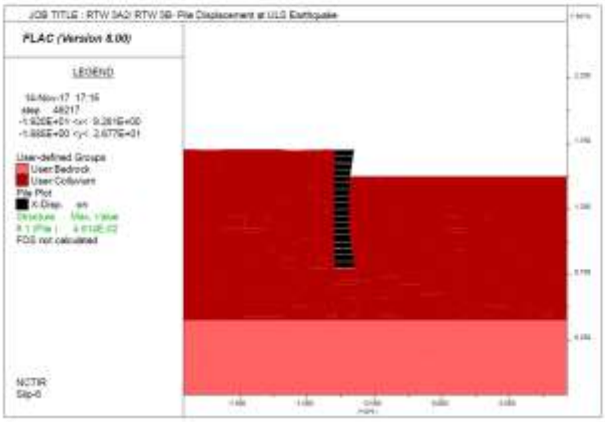

Figure 8- FLAC Model

In the FLAC analysis, all the sections and their respective construction sequences were modelled as per the construction programme that was provided by the Delivery Team (Construction crew). Usually such program was driven by the need to have a temporary road and rail passage open as quickly as possible, with further work undertaken at a later date to achieve the permanent cross-sections and topographic profiles.

The structural demands such as bending moment, shear forces, deformations of the wall and settlement of the road and rail corridors were estimated by and inferred from the FLAC models. The results from these models was also used to optimize the spacing of the bored piles, pile size and propping systems such as anchors and concrete slabs The slope stability in both static and seismic design cases also were analysed using the FLAC models and check make for ensure the design criteria.

Site observations after the Kaik ura earthquake sequence indicated the slopes above the site had experienced significant slope failures and the existing seawall and slopes below the road surface had generally performed very well. The new Site 6 RTW 3 structure has been designed to increase the geotechnical performance of the slopes below the road surface in future earthquakes, and is assessed as unlikely to exhibit significant damage after a future ULS seismic event.

An observational approach to the design of RTW 3 and the adjacent seawall indicates that the proposed design will improve the geotechnical performance of Site 6 without trying to restrain the whole slope, which would have been prohibitively expensive. i.e. the road and rail platforms have been designed to act as a single sliding block, and, is expected to exhibit high levels of seismic performance.

A risk of debris flow and / or rockfall damage due to failure of slopes high above the road and rail platforms has not been addressed in this paper. Mitigation of the risk which is associated with upslope rockfall and debris flows has been addressed by the NCTIR Slope Hazards Team via the design of catch structures, division channels and bridge structures.

8. CONSTRUCTION SEQUENCES

The following construction sequences were developed and recommended to maximise safety levels at all stages of the wall construction.

8.1 Bored Pile Retaining Wall

- Complete all slope stabilisation works as appropriate, above the RTW 3 construction site.
- Construct bored pile – Provide a temporary casing to all pile excavations which extends 1.0 m minimum above the adjacent working platform surface.
- Construct capping beam.
- Complete excavation in front of the wall in 1.0 m deep stages; Construct the wall lagging and drainage as appropriate after each excavation stage.
- Construct handrails, drainage controls.
- Install architectural dressing to front of the wall.

Figure 9- Bailey bridge installed above the bored pile retaining wall to enable traffic to pass safely, during the RTW 3 and debris flow bridge construction

9.2 Bored Pile Retaining Wall with Ground Anchor

- Complete all slope stabilisation works as appropriate, above the RTW 3 construction site.
- Construct temporarily bailey bridge crossings adjacent to RTW 3 (see figure 9)
- Construct bored pile - Provide a temporary casing to all pile excavations which extends 1.0 m minimum above the adjacent working platform surface (see figure 10).
- Pre-fabricate ground anchor unit and grout internal annulus over full bonded length.
- Drill anchor hole and install anchor assembly. (Grout was pumped in to the hole via grout tubes from the bottom and pumped upwards, to prevent trapped air bubbles).
- Grout internal annulus below ground level.
- Grout external annulus over full anchor length.
- Construct capping beam.
- Grout internal annulus with-in the capping beam.
- Test anchors when anchor and capping beam grout and concrete had achieved the specified minimum 28 day strength.
- Tension and lock off the anchors at the specified prestress load.
- Complete excavation in front of the wall in 1.0 m deep stages; Construct the wall lagging and drainage as appropriate after each excavation stage.
- Construct handrails, drainage controls.
- Check anchor prestress loads, install anchor end caps and grease packs.
- Install architectural dressing to front of the wall.

Figure 10-Bored pile construction (at RTW 3D1, an anchored section of RTW 3

9. SAFETY IN DESIGN CONSIDERATIONS

The following features were considered and incorporated into the design of RTW 3 to improve the safety of the structure during construction and for the end users.

- Bored piles were adopted, amongst other reasons to minimise the live rail disruptions and risk to rail operations once the temporary rail reconnection have been made.
- A concrete nib was provided on the top of the capping beam to prevent ballast from falling into SH1, and, to reduce the risk of tool fall from KiwiRail maintenance activities.
- The capping beam was designed to provide a safe working platform for the KiwiRail maintenance workers
- A sturdy safety handrail / barrier was provided on the outside edge of the capping beam to provide fall protection for future maintenance personnel.
- A temporary steel casing was specified by the designers which extended 1.0 m minimum above the ground surface to improve the safety of the construction workers and avoid any pile hole collapse.

Figure 11-State Highway 1 opened after road debris flow bridge construction, the debris flow channel construction underway.

11. CONCLUSION

NCTIR Site 6 was severely damaged by landslides originating above the road surface during the November 2016 Kaik ura Earthquake.

The overarching project goal of reopening SH 1 and the MNL coastal corridors as quickly and safely as possible was a key consideration during the options assessments phase of this project and ultimately contributed to the selection of the preferred solution for RTW 3.

The bored pile option, although expensive compared to other options was developed in order to open the road with minimum disruptions to the existing road and rail networks. It was also considered the best in terms of constructability and providing a robust durable solution in a coastal environment.

The proposed bored pile solutions create a stable transport infrastructure for any future earthquake events.

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REFERENCES

AASHTO(2016)-LFRD Bridge Design Specifications, Seventh Edition 2014 with 2016 Interim Revisions.

Civil Engineering Standard, Retaining Walls, Issue 1, 30-06-2016, KiwiRail

Eurocode 8-Part 1EN 1998-1:2004+A1:2013 and Part 5 EN 1998-5:2004.

FLAC, Version 8.0.448, Copyright © 2016 Itasca Consulting Group, Inc.

Fault Rupture Map- https://www.geonet.org.nz/news/6JadI02Pi8YUAOiiKUOgO2

Google Earthpro

Kiwirail Civil standards for Retaining walls (C-ST-RW-4104) and Ground Engineering and Earthworks(C-ST-GE-4105).

NCTIR-https://www.nzta.govt.nz/projects/kaikoura-earthquake-response/

NZTA, 'Bridge Manual (SP/M/022),' third edition, Amendment 2, May 2016.

Rattenbury, M.S.; Townsend, D.; Johnston, M.R. (compilers) 2006: Geology of the Kaikoura area: scale 1:250,000 geological map. Lower Hutt: GNS Science. Institute of Geological & Nuclear Sciences 1:250,000 geological map 13. 70 p. + 1 folded map

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