KAIKŌURA EARTHQUAKE RECOVERY, DESIGN OF A 13M HIGH GEOGRID REINFORCED, NO FINES CONCRETE GRAVITY SEAWALL IN A HIGH SEISMICITY ENVIRONMENT

S.B. Glue
BE(Civil) Hons, MEngNZ, NZGS

ABSTRACT

The North Canterbury Transport Infrastructure Recovery (NCTIR) Alliance was formed to deliver the repairs to the national road and rail transportation corridors after the Mw 7.8, Kaikōura earthquake which occurred on 14 November 2016. At Ōhau Point, located approximately 26 km north east of the Kaikōura CBD via road, approximately 240,000 m^3 of landslide debris buried the rail, road and adjacent coastline.

Between February 2017 and July 2018, a new 900m long seawall was designed and constructed as part of the coastal realignment of State Highway 1 (SH1) around Ōhau Point. This structure incorporates mechanically stabilised earth comprising mainly cement stabilised backfill with geogrid reinforcing, and five-ton concrete block facing.

The most complex section of this seawall is around a rock outcrop known locally as Shag Rock where the seawall is up to 13m high. The constraints and challenges in this area include maintaining access along the existing SH1 above the wall and ecological constraints. A special complex no fines concrete gravity wall (NFC G-Wall) was designed and constructed to buttress the slope and older seawall. This structure, and the wider fill and earth platform which supports the widened roadway, is designed to slide as a block under 0.76g peak horizontal ground acceleration.

This paper presents the results of the two-dimensional FLAC modelling which was completed to analyse and design the 13m high geogrid reinforced NFC G-Wall at NCTIR site 6. It also describes the pragmatic observational approach which was taken for the seawall design, highlighting the seismic sliding mechanism and issues that arose during design and construction of the seawall.

Keywords: mass, gravity, retaining wall, no fines concrete, seawall, Kaikōura, earthquake, FLAC, seismic, displacement, sliding, NCTIR, observational.

1 INTRODUCTION

The Mw 7.8 Kaikōura earthquake occurred on 14th November 2016 at 12.02 am. This earthquake initiated movement on several known faults towards over a total distance of 170 km. Figure 1 below shows the faults which are currently known to have ruptured as a result of the 2016 Kaikōura earthquake.

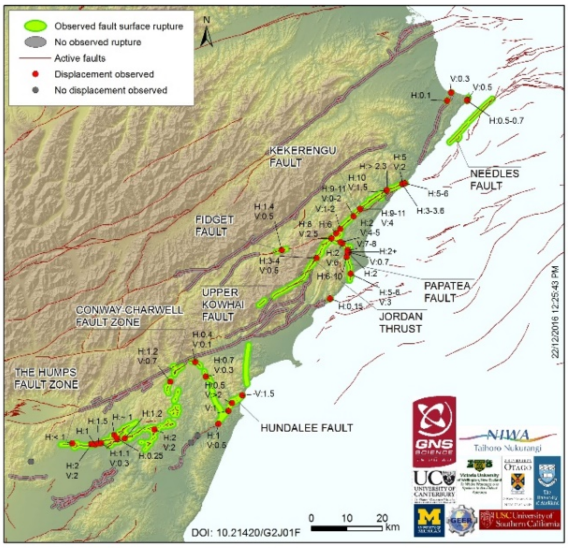

Figure 1: 2016 Kaikōura earthquake fault rupture map

The Kaikōura earthquake caused significant damage to the North Canterbury and Southern Marlborough road and rail transportation routes, in particular on Inland Route 70 (IR70), State Highway 1 (SH1) and the KiwiRail Main North Line (MNL). Of particular note, the Kaikōura earthquake and following storm events triggered hundreds of landslides and slips along the transport corridors which required the removal of over one million cubic metres of colluvium and rock fall debris.

The North Canterbury Transport Infrastructure Recovery (NCTIR) project is an alliance partnership between Downer New Zealand, Fulton Hogan, HEB Construction, Higgins, and the New Zealand Government which is represented by the New Zealand Transport Agency (NZTA) and KiwiRail. The primary objective of the NCTIR alliance is to re-establish and restore the damaged national transport networks and create a resilient transportation network along IR70, SH1 and the MNL railway, and keep traffic moving on alternate routes.

The Kaikōura earthquake and following storm events triggered a number of landslips along the Kaikōura Coastline. This paper pertains to part of an area named Site 6 by the NCTIR team, approximately 26 km north of Kaikōura (via road), and 0.5 km south of Ōhau Point. At Site 6 the earthquake and following storm events triggered landslides and rock falls resulted in approximately 240,000 m^3 of colluvium and completely buried the SH1 roadway and the rail tunnel portals. As a result of the Kaikōura earthquake the Ōhau Point foreshore was uplifted by between 1 and 2 m.

Figure 2 below provides a location plan for Ōhau Point and Figure 3 shows an aerial photograph of the area south of Ōhau Point after the 2016 Kaikōura earthquake annotated to show the approximate alignment of the original and new SH1 alignment.

The MNL alignment runs between the base of the Kaikōura mountain range and SH1. Immediately south of Ōhau Point the MNL runs into Tunnel 19 and bypasses most of Ōhau Point. The southern portal to Tunnel 19 was buried in landslide debris during the earthquake and the new rail alignment is 5 m closer towards the sea to enable rock fall protection measures to be constructed upslope.

Within the scope of the NCTIR works programme, SH1 was realigned around Ōhau Point and moved to a lower elevation and closer to the sea to allow for upslope rock fall protection measures to be constructed along the original SH1 roadway. To tie the new SH1 road alignment into the pre-earthquake road alignment, enable new rail rock fall protection measures to be constructed and allow the construction of a new debris flow bridge south of Ōhau Point, additional realignment of the road was required around a rock outcrop feature known locally as Shag Rock.

In the Shag Rock area, a short section of seawall and slope supporting the road south of the Site 6 zone did not fail during the Kaikōura earthquake. However, earthworks and a no fines concrete gravity seawall (NFC G-Wall) were required to be constructed below the proposed road either side of Shag Rock (between Ch 01S-0188-13340 and 13790) to tie into the adjacent section of seawall to the north and enable the road and rail realignment works to be constructed. Figure 4 below shows a typical cross section through the NFC G-Wall, SH1 and MNL north of Shag Rock.

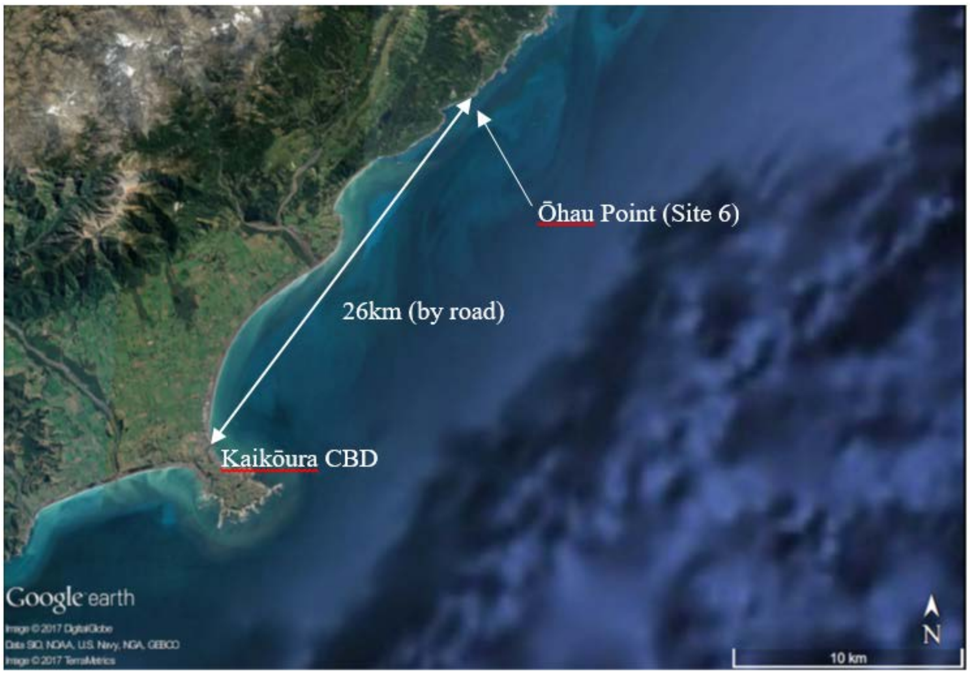

Figure 2: Site Location - Image from Google Earth Pro ©

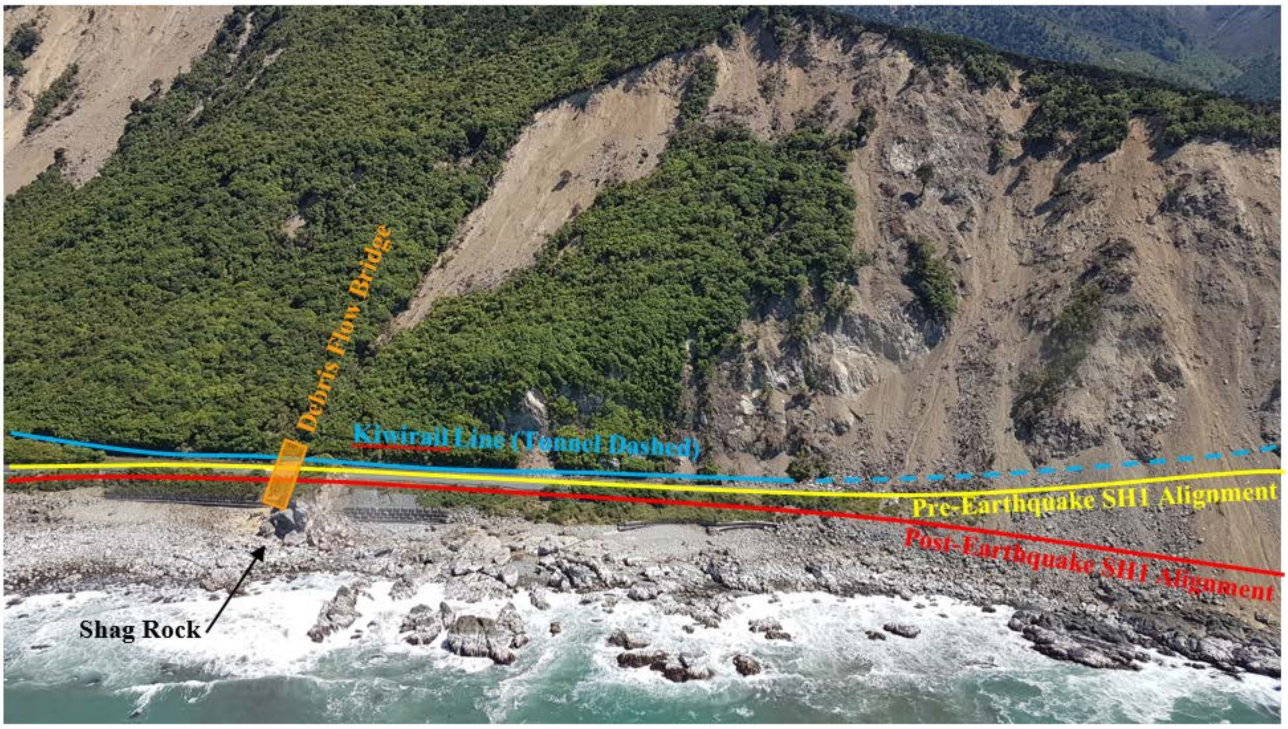

Figure 3: Photograph of area south of Ōhau Point (14 December 2016) showing earthquake related damage with approximate locations of pre-earthquake and post-earthquake road alignment

2 DESIGN CRITERIA AND PHILOSOPHY

Due account of the following physical constraints and preferred design philosophy was made during detailed design of the Site 6 NFC G-Wall:

1) The plan location was governed by the resource consent construction limit boundary (approximately 3 to 5 m seaward from the base of the original seawall blocks).
2) The seaward side of Shag Rock is a nesting site for shags, hence a sensitive ecological site and the proposed retaining structure had to be built around the existing rock outcrop in a manner which minimised visual and environmental impact.
3) The existing road and seawall north and south of Shag Rock needed to be maintained during construction to allow it to be used as a haul road by the delivery (construction) team.
4) Geotechnical investigations indicated that the site of the road behind the new seawall was underlain by up to 15m of colluvium, although the wall itself is founded on sandstone bedrock. Shag Rock, which is located in the middle of the proposed NFC G-Wall alignment, comprised a sandstone outcrop with rock virtually present from the existing ground surface. This indicated that the rock surface is near-vertical immediately north and south of Shag Rock, i.e. a "buried cliff" feature is present around Shag Rock.
5) The NZTA preferred a retaining structure which resulted in a controlled deformation failure rather than a structural overload failure. NZTA would accept an increased level of sliding deformation under ULS loading as long as the sliding block was designed to move as one intact mass.
6) Design was to meet the requirements of the NZTA Bridge Manual Third Edition (2nd Amendment) and any approved departures with a ULS seismic design requirement of PGA_H = 0.76 g.
7) This location generally exhibited good geotechnical performance during the 2016 Kaikōura earthquake sequence in that the existing seawalls did not exhibit any sign of seismic damage nor significant displacement as a result of the Kaikōura earthquake sequence.

2.1 DEPARTURE REQUESTS

With respect to horizontal displacement of the Site 6 NFC G-Wall during a design seismic event, a departure request was accepted by NZTA to allow up to 1.0 m of horizontal sliding during a ULS seismic event. This was accepted on the basis that the displacements would have limited disruption and relatively easy repair.

3 OPTIONS CONSIDERED

Due to the site constraints described previously in this paper, only the following three construction options were short listed and investigated for this location:

- Option 1: Full height anchored reinforced concrete pile retaining wall;
- Option 2: Short reinforced concrete piles with an MSE slope above, and;
- Option 3: No fines concrete gravity wall with geogrid reinforcing.

3.1 OPTION 1: FULL HEIGHT ANCHORED REINFORCED CONCRETE PILE RETAINING WALL

The first option investigated involved augering and grouting large diameter T shaped precast concrete piles into the bedrock in front of the existing seawalls and slopes. The new piled wall would be backfilled with either site won gravel, or no fines concrete and the piles would have anchors installed near the top of the piles to control wall deflections and reduce bending moments in the piles to acceptable levels. Between the piles, precast concrete facing panels were proposed which would have been provided with a surface finish to mimic the concrete block seawall design to the north and south of the site.

The engineering analysis which was completed to assess Option 1 highlighted that, as expected, the use of a stiff cemented backfill material significantly reduced the pile bending moments. However, the basal shear forces and anchor loads were still very high when the slope started to deform during a design (ULS) earthquake event. The required size of the piles and anchors to withstand the ULS design loads, and the risk of having a brittle structural failure during a large earthquake, meant that this option was deemed technically undesirable, further assessment also indicated that this option was economically unfeasible and as such was not investigated further.

3.2 OPTION 2: SHORT PILES WITH A MSE SLOPE

This option involved augering 7 to 10 m long, 1.2 to 1.5 m diameter reinforced concrete piles into the bedrock. This would allow a wall with a maximum retained height of 3 to 5 m to be constructed in conjunction with a 1H:1V geogrid reinforced MSE slope above the new wall to support the new road construction. Due to the boundary constraints at the bottom of the slope, and the resulting narrow construction corridor, preliminary engineering assessment indicated that there was not enough space available to enable the geogrid reinforced MSE wall to be constructed and keep the construction road above the site open.

In addition, the ULS seismic design loads resulted in very high shear force and bending moment demands in the base piles, and, significant deformation was predicted to occur within the MSE slope above the wall. Due to the inability of this option to enable the temporary haul road to remain open at all times during construction, the risk of adverse seismic deformations, the high level of stress at the base of the piled wall, and, the risk of a brittle failure mode during an earthquake, this option was not investigated further.

3.3 OPTION 3: NO FINES CONCRETE GRAVITY WALL WITH GEOGRID REINFORCING

The third design option, which was eventually progressed to detailed design and construction, was an NFC G-Wall. The no fines mass concrete was assessed to result in a high-performing stiff mass between the seawall blocks and the existing colluvium slope/old seawalls. Detailed engineering analysis indicated the base foundation slab of the wall needed to be keyed into the bedrock to create a sliding plane between the base slab and the no fines concrete mass.

Detailed analysis also demonstrated that the no fines concrete would provide mass and stiffness to the overall slope and improve the static and seismic performance of the existing slopes and seawalls. However, once the colluvium started to deform during a strong earthquake, the modelling indicated that the no fines concrete block would likely to start to slide and move globally with the whole slope. As previously described, the existing slope and seawall did not show any signs of tensional deformation or displacement as a result of the Kaikōura earthquake sequence and the overall design philosophy is expected to increase the geotechnical stability and seismic performance of the overall slope.

3.4 PREFERRED OPTION

Option 3 was assessed as likely to address all five of the design constraints described above and as such was progressed to detailed design and construction. Figure 4 below presents a typical cross section through the subject site and shows the preferred seawall option and other noteworthy structures around the new seawall.

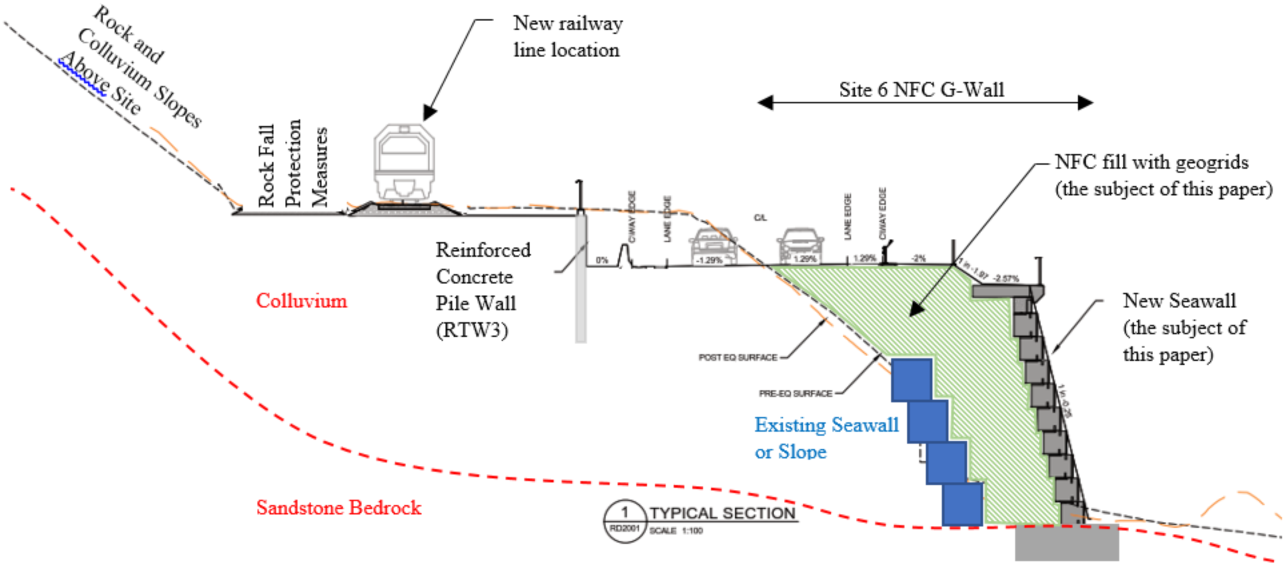

Figure 4: Typical cross section through the Site 6 NFC G-Wall, SH1 and railway line

4 WALL DESIGN

4.1 BASIS OF DESIGN

Given the complex nature of the NFC G-Wall components and the structure/rock/soil interactions, the two dimensional finite difference software package FLAC was used to analyse the seawall structures for the NCTIR alliance.

4.1.1 Geological conditions

The bedrock at Site 6 comprises Pahau Terrane (Ktp). This unit is described on the relevant published geological map (Rattenbury 2006) as well bedded and poorly bedded sandstone. These units are of early Cretaceous age and are approximately 145 million years old. A thin veneer of beach gravels and/or colluvium, up to 15 m thick overlies the sandstone bedrock in most locations within the proposed NFC G-Wall construction footprint.

4.1.2 Seismic design

Immediately following the 2016 Mw 7.8 Kaikōura earthquake, the damage at Site 6 to the road and rail infrastructure was observed to be predominantly due to landslip debris originating from the slopes above these assets. The estimated peak horizontal ground acceleration at the site during the Mw 7.8 Kaikōura earthquake is inferred from data published by the USGS as being the order of 0.50 g. 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.

The design ULS horizontal seismic acceleration (PGA_H) calculated using AS/NZS1170.5 is 0.76g for an Importance Level 3 (IL3) structure and a 1/2500-year return period event. A vertical seismic acceleration (PGA_V) value of 0.31g (approximately 40% of the horizontal PGA_H value) has been used in the pseudo-static analysis.

4.1.3 FLAC design parameters

The colluvium is assessed from the relevant investigation data to be a mix of silt, sand, gravel and boulder material up to 2 m diameter. The corresponding field testing yielded uncorrected SPT N values of between 4 and 50+. As such this material was assessed as being equivalent to a medium dense soil with some cohesion. With respect to this assessment it should be noted that the existing colluvium slopes have a surface slope of up to 45° and survived the 2016 Kaikōura earthquake sequence with no noticeable damage. Back analysis of the relevant existing slopes indicates the constituent soil material conservatively has an average friction angle of 35° and an average cohesion of 5 kPa.

The no fines concrete is designed as a low strength concrete with a design UCS strength of approximately 2 MPa. Testing during construction indicated that the 28-day UCS strength testing varied from 2 to 8 MPa.

The geotechnical design parameters which were adopted during the FLAC analysis are summarised in Table 1 below:

Table 1: FLAC analysis geotechnical design parameters

Name	Soil model	Unit weight (kN/m^3)	Cohesion (kPa)	Tension (kPa)	Friction angle (°)	Youngs modulus (MPa)	Poisson's ratio
Sandstone bedrock	Modified Hoek-Brown	2550	GSI = 30, m_i = 15, σ_{ci} = 50MPa			815	0.2
Colluvium	Mohr-Coulomb	1800	5	0	35	30	0.3
No fines concrete	Mohr-Coulomb	1800	130	330	42	22000	0.2
Concrete blocks	Mohr-Coulomb	2550	850	2840	35	37200	0.2
MSE fill	Mohr-Coulomb	1990	0	0	38	130	0.3

The geotechnical design parameters which have been adopted in the FLAC models for the interface between the soil and structural elements are summarised in Table 2 below:

Table 2: FLAC analysis soil – structure interface design parameters

Interface	Model	Type	Friction angle (°)	Cohesion (kPa)	Shear modulus (MPa)	Bulk modulus (MPa)
Colluvium – filter fabric – no fines concrete	Mohr-Coulomb	Unglued	27	0	20000	20000
Concrete – no fines concrete	Mohr-Coulomb	Unglued	35	0	20000	20000
Concrete - concrete	Mohr-Coulomb	Unglued	35	0	20000	20000
Concrete - rock	Mohr-Coulomb	Unglued	35	0	20000	20000

4.2 INITIAL DESIGN

The design and construction program within the NCTIR alliance was extremely fast and reactive during the first year of construction. This was because its primary objective (rightly so) was to reconnect the national road and rail corridors as quickly as possible. The final road and rail alignment for the section of seawall around Shag Rock and the adjacent debris flow bridge was not finalised until late in the design process and after construction had already started for the main portion of seawall around Ōhau Point. This left only two weeks for optioneering, initial design and the issue of the first pack of construction drawings to meet the project construction deadlines.

For the initial design analysis, the FLAC model was only pushed to maximum seismic yield capacity (approximately 0.4g pseudo-static PGA_h) to confirm the whole slope ultimate limit state (ULS) failure mechanism. The 0.4g pseudo-static PGA_h is approximately equivalent to half of the ULS design peak horizontal seismic acceleration which is a commonly accepted PGA reduction factor for pseudo-static stability analyses. This showed that under high seismic loads the wall and slope behind started to slide as an intact block and met the design requirements.

4.3 DETAILED DESIGN

Detailed design of the NFC G-Wall was completed a few months after the construction of the NFC G-Wall had commenced. As previously described, during the Kaikōura earthquake sequence, the slopes above the road surface exhibited significant slope failures and the existing seawall and slopes below the carriageway performed very well. The proposed NFC G-Wall was assessed to improve the seismic performance of the existing lower slopes in future earthquakes and it was highly unlikely that the colluvium slope below the carriageway would exhibit significant displacement without the impact of a major slope failure above the site. An observational approach to the wall design indicated that the proposed works would improve the seismic performance of the slope below the carriageway without having to restrain the whole slope (which would be prohibitively expensive).

To model the seismic effects on the wall, the earthquake load was applied in incremental steps for both the horizontal and vertical components of the design seismic acceleration. The horizontal incremental load steps were set at 0.05 g and the model was run for up to 60 minutes to obtain stability or until failure occurred. This method was developed by Ioannis Antonopoulos with the assistance of ITASCA for the analysis of seawalls for NCTIR, and is part of discussion presented in other papers under publication.

The FLAC model results indicated that the mode of failure for the wall is a mass block sliding mechanism between the base of the wall and the foundation slab. Such failure mode is the preferred mechanism as it is expected to result in lower risk of significant seismic damage to the wall and road surface, and, such damage should be easily identifiable and repairable after a significant seismic event.

The results of the detailed FLAC analysis indicated that the new seawall block wall is stable statically and is not expected to exhibit significant movement up to approximately an SLS level earthquake. However, at higher levels of seismic acceleration the Site 6 NFC G-Wall is expected to start developing high stress concentrations and a tension/shear failure through the no fines concrete block in particular over the bottom 3 m of the wall. This is expected to lead to the no fines concrete losing a significant portion of its cohesional and tensional capacity. The existing FLAC model setup can't reliably simulate the soil behaviour after this point of no fines concrete failure but running a static model with gravel like parameters indicates that the bottom blocks would bulge out and the wall would start to crush. This mode of failure would not be acceptable, as there is a risk of rapid wall collapse at around a SLS level event. As such, the final design for the NFC G-Wall incorporates a toe buttress to mitigate this issue.

The FLAC model output for the XY stresses for initial design (wall configuration 1) is shown in Figure 5 below:

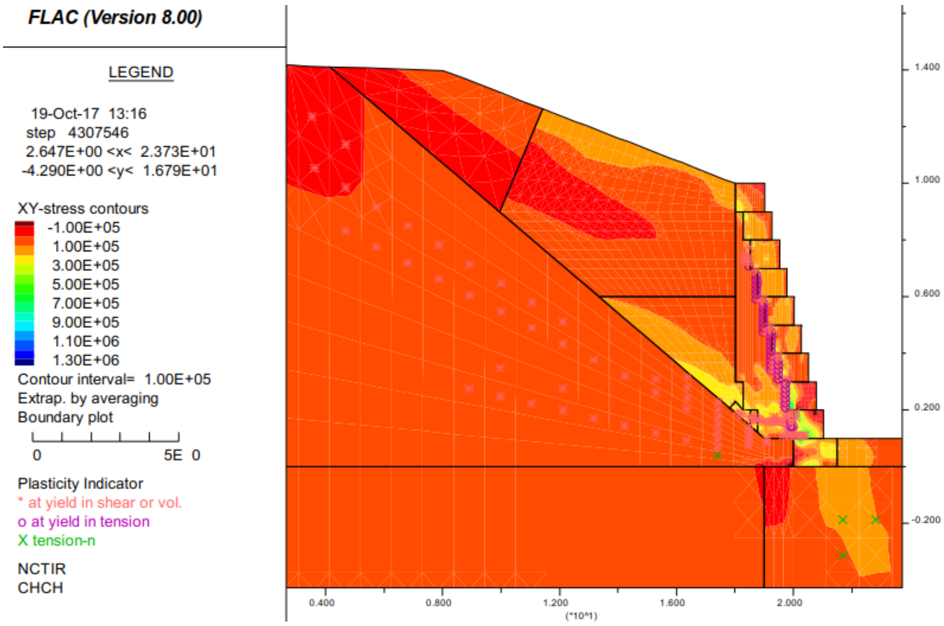

Figure 5: FLAC XY stresses output for wall configuration 1 and 0.15 g PGA$_h$ pseudo-static seismic load

To fix the issue with stress concentrations in the bottom 3m of the no fines concrete, an option of installing a 3m by 3m triangular stepped concrete buttress over the bottom 3m of the wall was modelled in FLAC. This analysis indicated that the concrete buttress effectively transferred and spread the majority of the load from the upper sections of the wall and limited the stresses in the bottom of the no fines concrete to acceptable levels.

With the concrete buttress detail, the wall is still expected to "fail" through sliding along the base of the wall between the no fines concrete/buttress and the foundation concrete. The foundation concrete is keyed into rock and the buttress was detailed to be a construction joint with no structural connection between the buttress and the foundation.

If the buttress were to become locked in by a rock outcrop or material in front of the wall, the wall could still slide through the no fines concrete and on top of the buttress. To account for the sliding failure mechanism a 1m wide sliding surface was provided in the top of the buttress configuration.

Table 3 below summarises the seismic yield acceleration for different wall layouts analysed in FLAC.

Table 3: Estimated seismic yield acceleration values for the three different FLAC model cases

Wall configuration	FLAC model case	Yield acceleration a_c*
1	Soil slope with no existing seawall blocks, no base buttress	0.15 g
2	Soil slope with no existing seawall blocks, 3m high base buttress	0.45 g
3	Existing concrete seawall blocks with slope above, 3m high base buttress	0.45 g

* Unidirectional pseudo-static acceleration

The FLAC model output for the XY stresses for wall configuration 2 is shown in Figure 6 below:

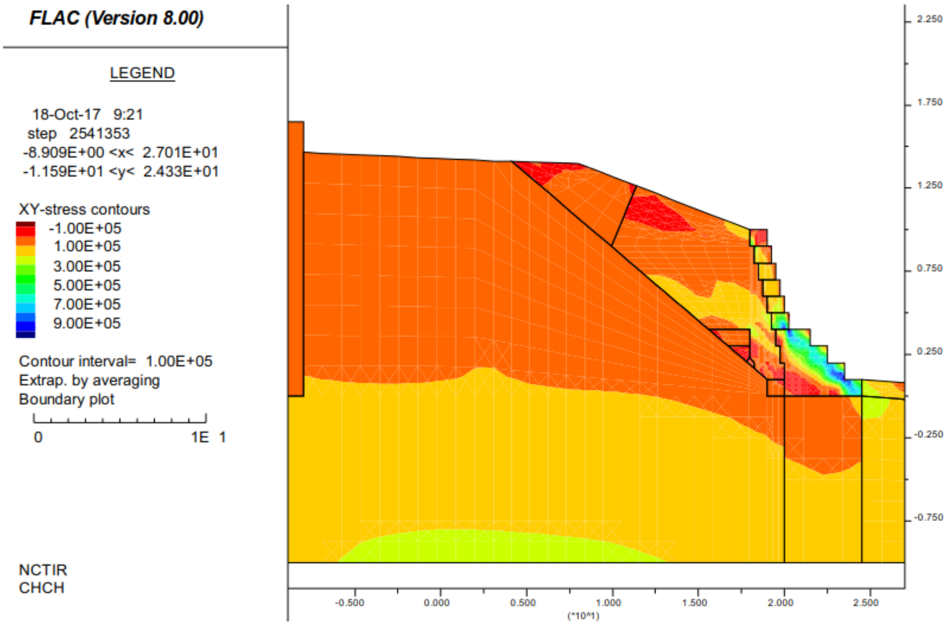

Figure 6: FLAC XY stresses output for wall configuration 2 and 0.45 g PGA_h pseudo-static seismic load

4.3.1 Displacement assessment

The displacement predictions from the pseudo-static seismic analysis typically peak out at between 100 and 600 mm before the models reach “failure”. In order to estimate the maximum seismic displacements as a result of ULS seismic shaking, we have compared the results of the pseudo-static seismic analysis to empirical seismic slope displacement estimates. The yield acceleration is assumed to be the pseudo-static seismic acceleration at which the FLAC models reach failure.

These empirical displacement estimates were determined using the methods published by Bray & Travasarou, Jibson, and, Ambraseys & Srbulov. These methods compare the yield acceleration (seismic acceleration for FOS= 1.0) to the design acceleration for a seismic event of a specific magnitude.

The above analysis methods are based on empirical data and observations after earthquakes with a moment magnitude (Mw) of between 5.3 and 7.6. To account for potentially high vertical accelerations, the displacements summarised in the following analysis have a 5% probability of exceedance. Figure 7 below presents a summary of the design chart which formed the basis of the estimates of embankment displacement due to seismic acceleration.

The Bray & Travasarou method also considers the initial fundamental period of the proposed Site 6 NFC G-Wall site and the moment magnitude of the design earthquake. For the purposes of these estimates the gravity wall was assumed to be 13 m high and founded on Class B Rock with an average shear wave velocity of 360 m/s. An initial fundamental Period of 0.14 seconds was estimated using the following equation:

$$Ts = \frac{4H}{Vs} = \frac{4x13}{360} = 0.14\ seconds \tag{1}$$

The Ambraseys & Srbulov method also considers the focal depth and the distance to the source/epicentre of the earthquake. The earthquakes used to formulate the Ambraseys & Srbulov method had an average focal depth of 10 km and a distance of 25 km. The formula is relatively sensitive to focal depth/epicentre distance. The closest major fault (requiring a near fault factor consideration) is the Hope Fault which extends out to sea approximately 2 km south west of the site. A focal depth of 5 km and a distance of 2 km to the source/epicentre has been assumed in the analysis for this fault scenario. The resulting curve is greater than that the Jibson and Bray & Travasarou methods estimate (refer to Figure 7).

A yield acceleration of approximately 0.45 g, which was estimated from the pseudo-static FLAC seismic analysis, allows for a yield ratio of 0.60. Using this yield acceleration results in a prediction of between 50 and 600 mm of displacement in a design ULS earthquake. This is in general agreement with the FLAC displacement predictions around the pseudo-static yield acceleration.

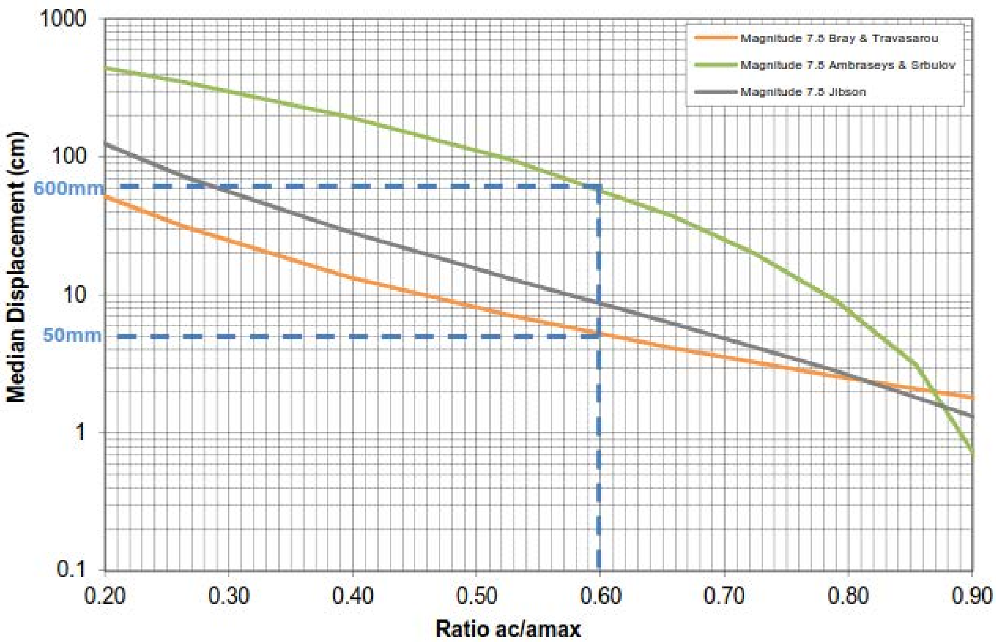

Figure 7: Estimated Displacements with 5% Probability of Exceedance for a 13 m high wall on Class B Rock and a 7.8 Moment Magnitude Earthquake

5 CONSTRUCTION PHASE

The following construction sequences were recommended and utilised during construction as a result of the detailed geotechnical analysis:

- Excavate and pour 30 MPa mass concrete foundation concrete base pad keyed a minimum of 0.6 m into bedrock with a 0.5 m deep x 45° shoulder load distribution pad in front of the wall (dependent on rock depth).
- Place modular concrete seawall blocks and backfill with no fines concrete and Tensar RE580 geogrid reinforcement.
- Above the final seawall block construct an in-situ pour block and install the capping beam in place. The capping beam forms the shared-use path.
- Install continuous geogrid layer across road pavement to span between no fines concrete and colluvium sub-grade surface and minimise the risk of differential settlement.
- Construct 3 m x 3 m stepped 30 MPa concrete buttress in front of the lower 3 blocks. (i.e. buttress the lower 3 m of the NFC G-Wall)

6 CONCLUSIONS

The following conclusions are drawn from the Site 6 NFC G-Wall design:

- No fines concrete is a self-supporting backfill material that can be used to reduce static and seismic loads on retaining structures. The use of no fines concrete also results in a highly effective drainage medium. In some situations, the use of no fines concrete can be more cost effective than compacted granular backfill in particular

in difficult access areas. However, care must be used when modelling such backfill material to ensure the no fines concrete material doesn't yield. After yielding and cracking the material would revert to pea gravel strength parameters.
- Odd shaped mass gravity retaining walls can work well where existing slopes are generally stable. However, care should be used when base widths become very narrow as bearing stress concentrations can become very high resulting in an undesirable failure mechanism.
- Observations after the Kaikōura earthquake indicates highly variable colluvium can perform better than indicated by traditional Mohr-Coulomb models.
- Designing retaining structures to resist very high seismic accelerations can lead to very conservative design solutions. Designing for higher allowable displacements and desirable "yielding" or "block-sliding" failure mechanisms is assessed to be a pragmatic and cost-effective philosophy to design complex retaining structures for major infrastructure projects.

Figure 8 below shows the aerial photograph of the NFC G-Wall site constructed to full height with capping beam installation underway.

Figure 8: Aerial photograph of 80% complete NFC G-Wall site (photograph taken 10 April 2018)

7 ACKNOWLEDGEMENTS

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