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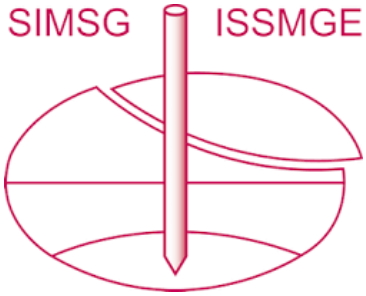

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SH1 realignment – Optimisations and innovations on new MSEW seawalls

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ABSTRACT: State Highway 1 north of Kaikōura has been adversely affected by the slope failures as a result of the magnitude 7.8 Kaikōura Earthquake in November 2016. Three major realignments are designed and constructed to minimise the effects of future slope failures and to provide a safe corridor. These comprise Mechanically Stabilised Earth Seawalls (MSEW) that are designed to withstand the high earthquake demands and the ocean forces. A series of innovations have been introduced in both materials and modelling techniques to optimise material quantities, speed of construction, and whole of life performance. The geotechnical modelling carried out takes into consideration the full structure, i.e. the MSE part and the concrete segmental facing. The full Soil (MSE, backfill, and natural ground) – Structure (segmental facing constructed by concrete blocks) Interaction has been considered under static and quasi-static. The quasi-static loading and the long-term performance are further optimised using innovative modelling techniques.

1 INTRODUCTION

1.1 *General*

The Mw 7.8 earthquake that occurred on 14th November 2016 at 12.02 am south of Kaikōura caused significant damage to the North Canterbury and Southern Marlborough road and rail transportation routes, in particular on State Highway 1 (SH1) and the KiwiRail Main North Line (MNL).

The event caused a regional uplift of the coastal area in the order of 1 to 2m, triggered about 80 landslides and rockfalls of significant size along the Kaikōura coastline, and caused significant damage to the coastal transport infrastructure, rendering approximately 20 km of State Highway (SH) 1 north of Kaikōura unusable and closed to the public for over a year, isolating many coastal and rural communities overnight.

North Canterbury Transport Infrastructure Rebuilt (NCTIR) Alliance was formed to re-establish and Seawalls, revetments, bridges and tunnels were all considered on a case-by-case basis as potential solutions to by-pass the largest landslides and restore the damaged transport networks.

A decision to build Seawalls in order to relocate sections of SH1 seaward and away from the potential landslide impact footprints around the three critical areas of Irongate stream and north and south of Ōhau Point a coastal realignment was decided on the basis of cost, material availability, reopening targets, and a desire to minimise the work's coastal footprint and associated environmental impact. The extents of the Seawalls are shown in Figure 1.

Figure 1. Location plan

1.2 *Concept*

The Seawalls have been designed as semi-flexible structures, capable of providing high levels of geotechnical and structural performance in future seismic events. They incorporate Mechanically Stabilised Earth Walls (hereafter referred to as MSEWs) with mass concrete block facing units (hereafter referred to as Seawalls). The Seawalls cover a total length of approximately 2.5 km and exceed 10.0 m in height in places.

A 2m length times 1m height times 1m depth custom precast concrete block was developed to increase the durability of the finished structure and allow a high degree of installation precision and consistency while being relatively easy to construct.

The 5-tonne blocks are structurally independent of one another to permit flexibility of the structure as a whole, relying on the geogrid and associated dowel connection to hold them in place during a future earthquake event. Approximately 7000 Seawall blocks were required for 2.5 km of Seawalls.

A typical detail of the Seawall showing the MSEW and facing block arrangement is presented in Figure 2.

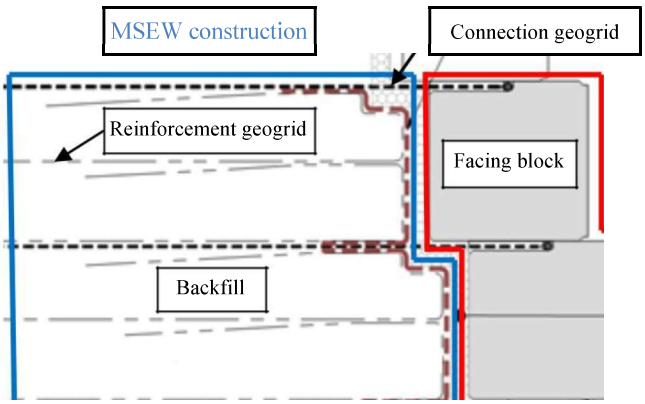


Figure 2. Typical seawall detail

2 DESIGN PHILOSOPHY

2.1 *Seismic performance criteria*

The basis of the seismic performance criteria specific to the Seawall design and performance are stipulated in the NZTA Bridge Manual Edition 3 (hereafter referred to as BM3). Additional criteria have also been agreed through departure requests. The Seawalls are of Importance Level 3, are built on Site Class B, they are expected to have a 100 year design life, and the seismic loading cases and included criteria are described below.

Minor Earthquake Event (α_{max}=0.19g): Factor of Safety against sliding ≥1.0 with no significant deformation of Seawalls. No disruption to traffic.

Design Level Earthquake Event (DE α_{max}=0.76g): Controlled sliding deformation (<300 mm) to occur prior to the propagation of plastic/ shear strains and/ or yielding resulting in internal instability. Up to 500 mm subsidence of carriageway behind reinforced zone. Carriageway remains trafficable.

Major Earthquake Event (α_{max}=1.14g): Further controlled sliding deformation (<1.0 m) to occur prior to the propagation of plastic/ shear strains and/ or yielding resulting in internal instability. Surround-ing environment considered to render carriageway unusable prior to internal failure of the Seawalls.

2.2 *Structural response*

The structural response of the coastal structures is summarised below against the actions to be considered as part of the design

Static loading: To resist the applied actions (traffic and dead load of structure) through the internal stability of the reinforced backfill and self-weight of the mass concrete facing blocks.

Seismic loading: Initially counteracted by the self-weight of the Seawall up to the critical acceleration, at which the Seawalls begin to displace horizontally. The magnitude of horizontal de-formation increases with increasing acceleration, with the design accelerations reached prior to the propagation of internal plastic/ shear strains and/ or internal yielding resulting in internal instability.

Wave action: Initially counteracted by the rock toe berm and/ or self-weight of the mass concrete facing blocks which are further restrained by the passive pressure of the reinforced backfill mass. A custom 45° nosing built into the capping blocks also assists in reducing overtopping of the wall by deflecting wave run up under large wave actions.

Flood: The Seawalls preclude the build-up of water pressures within or behind the wall over and above the allowance for a 1.0 m head of water within the reinforced backfill (behind the facing blocks). Any potential increased water pressure on the rear of the facing blocks is resisted by the self-weight of the facing blocks and their connections to the reinforced backfill mass.

2.3 *Seawall design methodology*

In general, the Seawalls comprise mass concrete facing blocks connected to reinforced, stabilised backfill. The Seawall backfill comprises two different types:

Mechanically Stabilised Earth Wall with cement stabilised AP65 fill and layers of geogrid at 0.5 m vertical centres.

No Fines Concrete (hereafter referred to as NFC) backfill with layers of geogrid at 1.0 m vertical centres.

Traditionally, and in accordance with the BM3, a Load Resistance Factor Design (hereafter referred to as LRFD) approach is adopted for the design of MSEWs. However, due to the high seismic demands, the scale of the project and the significant potential for cost savings and efficiencies to be realised, a performance-based design (hereafter referred to as PBD) methodology incorporating the use of finite difference (hereafter referred to as FD), stress-strain analysis was adopted.

The following combined PBD design methodology is implemented for the design of all Seawalls as part of the project:

Reinforcement requirements. Incorporates strength, pull-out, and facing connections. The design is carried out in accordance with LRFD, AASHTO 2014 and modelled with FD analysis.

The LRFD analysis is undertaken in accordance with AASHTO 2014 utilising in-house spread sheets. Design checks are carried out for three limit states as defined by AASHTO (AASHTO: Service limit state, Strength limit state and Extreme event limit state) for the following:

- Pull-out resistance of reinforcement.
- Tensile resistance/ strength of reinforcement.

Holistic performance. Incorporates external stability – overturning, sliding, seismic performance modelled with FD, and stochastic modelling of long-term strength degradation.

2.4 *Holistic performance*

Finite difference (FD) stress-strain analysis is carried out using FLAC software by Itasca (FLAC is a two-dimensional explicit FD program for engineering mechanics computation. It simulates the behaviour of structures built of soil, rock or other materials that may undergo plastic flow when their yield limits are reached).

The seawalls are modelled with a discrete approach in which both the geotechnical and the structural elements are included, such that the complete Soil-Foundation-Structure Interactions (hereafter referred to as SFSI) are considered.

Static and quasi-static analyses are undertaken to assess internal and global stability, overall performance of the seawalls, and SFSI of the seawall with the adjacent materials.

The quasi-static analysis is undertaken implementing an incremental/ stepwise earthquake loading procedure to assess the propagation of internal and external deformation and ultimately the mode and magnitude of deformation. The incremental loading procedure is a function specifically developed for this project and use with FLAC.

The method provides a complete seismic demand vs. displacement record and makes it easy to identify yield, the displacement associated with the Design Event seismic coefficients and an indication of what is to be expected for larger events.

As a means of cross validation, additional displacement-based analysis is undertaken. In the case of the FD quasi-static analysis, where the analysis results in yielding (sliding deformation) at a horizontal acceleration less than the design horizontal acceleration, the earthquake-induced displacements are estimated using Newmark sliding body displacement methods based on empirical relationships proposed by the following authors:

a) Kavazanjian (1997);
b) Ambraseys and Srbulov (1995);
c) Wood and Elms (1990);
d) Jibson (2007);
e) Ambraseys and Menu (1988).

The above methods generally relate the magnitude of horizontal displacement of a rigid body to the ratio of the design horizontal acceleration (α_{max}) to the critical/ yield acceleration (α_c) through various formulations. In addition to the ratio of α_{max} to α_c the following additional input parameters are considered for the methods (b), (c), and (d):

The Ambraseys & Srbulov (1995) method 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 maximum depth of 25 km. The closest major fault (requiring a near fault factor consideration) is the Hope Fault which generally varies between <1 and 3 km. A focal depth of 5 km and a distance of 1 km to the source/ epicentre are assumed as part of the analysis;

The Wood and Elms (1990) method considers the peak ground velocity, which is taken as 1.3 times the peak horizontal ground acceleration (design acceleration) according to the method;

The Jibson (2007) method considers the earthquake magnitude (MW) as a variable.

2.5 *Facing blocks*

The facing blocks are susceptible to earthquake induced displacement, in excess of that of the stabilised backfill. As such, the facing blocks require a positive connection to the backfill to aid in limiting the differential displacement.

The connection detail comprises a mechanical connection between the facing blocks and strips of geogrid reinforcement that are embedded in or connected to the backfill mass.

The bespoke connection detail was preferred over a proprietary system due to cost efficiency, however, it also meant that a variety of geogrid products could be used in the detail. Full scale field trials were un-

dertaken to assess the capacity and stress-strain characteristics of the connection detail.

The specific connection detail is dependent on the backfill type. A HDPE geogrid was selected for use when the backfill to the seawall comprises no fines concrete and a polyester geogrid was selected for use with the traditional engineered fill (AP65) and/ or cement stabilised fill options.

The polyester geogrid connection detail is shown in Figure 3 and Figure 4.

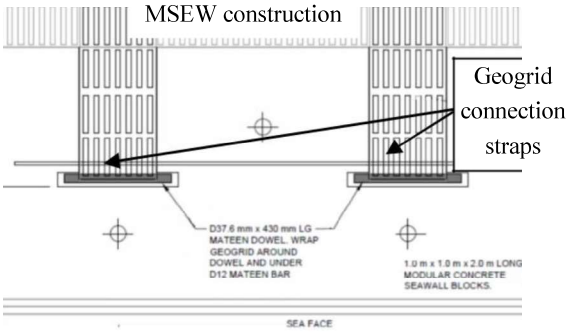


Figure 3. Polyester geogrid connection detail

Figure 4. Polyester geogrid connection photograph

3 GEOTECHNICAL DESIGN

3.1 *Geogrid Reinforcement Design*

The design of the geogrid reinforcement was undertaken during the early stages of the design and was based on a number of design features (wall face batter and fill type) which were subsequently changed to improve the seismic performance of the seawalls.

The adoption of PBD using FD analysis also resulted in a lower demand on the geogrid reinforcement. A comparison between the calculated maximum tensile forces (as per FLAC and AASHTO 2014 Extreme Event Limit State) that are mobilised in the layers of reinforcement per height of wall for a DE event is presented in the following figure 5.

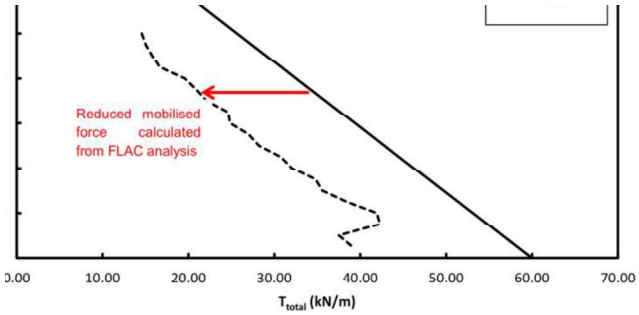


Figure 5. Maximum tensile force in reinforcement (Design Event)

The external geometry of the wall is a significant factor in achieving the performance criteria, specifically the batter angle of the facing, and the base width to wall height ratio. The batter angle of the facing was modified from a vertical face to a slope of 1H:4V to provide better holistic performance (combination of the stabilised fill and the concrete facing). The minimum base widths, and as such the minimum basal reinforcement lengths, required to satisfy the performance criteria are presented with wall height in Figure 7.

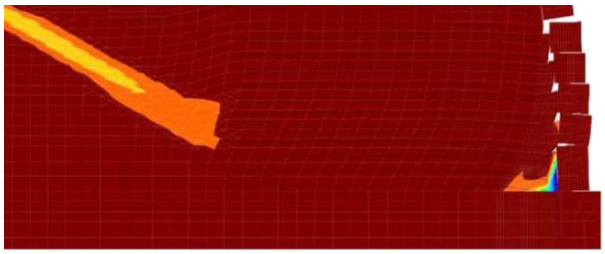
3.2 *Holistic Performance*

The fill stiffness and the external geometry of the wall are a significant factor in achieving the performance criteria, specifically the batter angle of the facing, and the base width to wall height ratio.

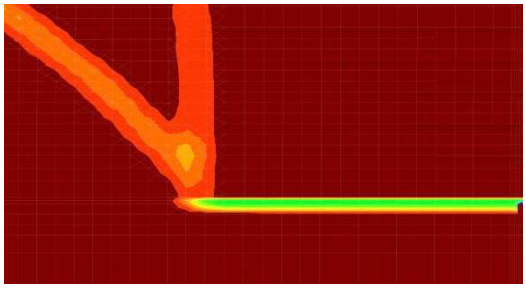
Initially the MSE was designed with traditional AP65 engineered fill. When both the geotechnical MSE elements and structural concrete facing were combined in one unified model it became apparent that the fill should be stabilised to maintain the expected performance under seismic loading.

The batter angle of the facing was also modified from a vertical face to a slope of 1H:4V to provide better holistic performance (combination of the stabilised fill and the concrete facing). The below figure 6 shows the difference in performance between the traditional AP65 fill and the cement stabilised fill respectively for a 7m high Seawall. The areas where shear strains are concentrated are visualized using the shear strain increments for a qualitative comparison.

Furthermore, the minimum base widths, and as such the minimum basal reinforcement lengths, required to satisfy the performance criteria were optimised and are presented versus wall height in figure 7.



(a) Traditional AP65 engineered fill



(b) Cement stabilised AP65 engineered fill

Figure 6. Model of a 7m high seawall – shear strain increments under DE showing the areas of strain concentrations

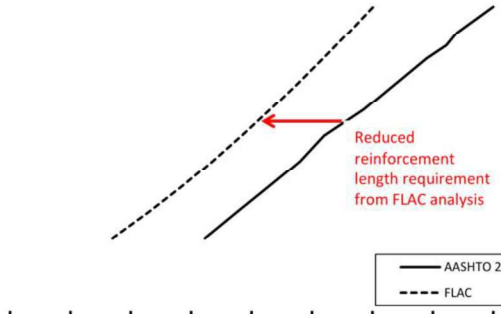


Figure 7. Reinforcement length requirements

To assess the long-term performance (over the 100 years design life) of the Seawalls, a degradation of both the shear and the tensile strength of the cement stabilised and NFC options was taken into consideration using stochastic modelling.

The stochastic modelling was carried out by applying a random distribution of degraded cohesion and tensile strength within the NFC mass. The stepped quasi-static approach was then used to assess the performance and failure threshold under earthquake loads.

The below figure 8 shows the distribution of the degraded tensional strength of a cement stabilised 9m high Seawall.

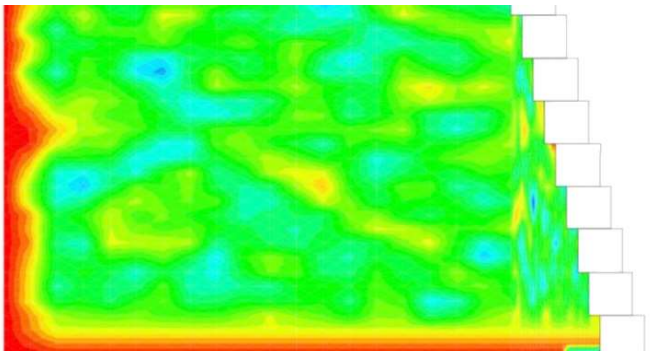


Figure 8. Model of a 9m high seawall – random distribution of tensile strength – the colour range is equally spaced from 0 kPa (red) to 45 kPa (blue)

4 DISCUSSION AND CONCLUSIONS

The overall project had strict milestones to allow for an early opening of the transport corridor, and to aid the local communities. These requirements and the earthquake demands resulted in a fast pace, intensive, innovative, and complicated work.

A series of innovations and optimisations were introduced and adopted into the design of the new Seawalls that form an essential part of the coastal realignment of SH1 north of Kaikōura.

A hybrid design methodology, incorporating LRDF and performance-based design finite difference analyses was further refined using stochastic modelling.

A bespoke connection detail between the concrete facing and the MSE was designed and introduced to maintain cost and construction efficiency.

5 ACKNOWLEDGEMENTS

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