Non-biennial Project 6UNI/575

An experimental study on geosynthetic reinforced soil walls under seismic loading

Report prepared for the New Zealand Earthquake Commission (EQC)

Project Leaders: Dr. Elisabeth Bowman
Assoc. Prof. Misko Cubrinovski

Principal Investigator: Perry Jackson

Department of Civil and Natural Resources Engineering
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Executive Summary

Background

Reinforcement of soil enables a soil slope or wall to be retained at angles steeper than the soil material's angle of repose. Geosynthetic Reinforced Soil (GRS) systems involve the incorporation of geosynthetic reinforcement (a strong and flexible engineering plastic) layered horizontally into select engineering backfill for the purposes of slope stabilisation and/or retaining slopes or walls.

GRS systems enable shortened construction time, increase seismic performance, potentially improve aesthetic benefits, and lower cost (by up to 50% less in one study) than their conventional retaining wall counterparts such as gravity and cantilever type retaining walls. They also meet many of the goals associated with sustainable development such as reduced carbon emissions and embodied energy, in addition to cost reductions noted when compared to conventional type retaining walls.

Motivation

Design of GRS in New Zealand is made using several different overseas standards and design guidelines (i.e. Federal Highways Administration, 2001; British Standards Institute, BS8006:1995; Australian Standard, AS4678-2002), in addition to manufacturers' own in-house design methods and guidelines (Tensar, for example). Hence, GRS structures in New Zealand have been built with varying resistance to static and seismic loads, and thus different seismic risk.

Experience in previous earthquakes such as Northridge (1994), Kobe (1995), and Ji-Ji (1999) indicate good performance of GRS under high seismic loads, in comparison to their conventional retaining wall counterparts. However, this good performance is not necessarily due to advanced understanding of their behaviour, but rather highlights the inherent stability of GRS against high seismic loads and conservatism in static design practices. Hence further experimental research is required to better understand seismic behaviour and to evaluate earth pressures, displacements and/or deformation under high seismic loads to advance design practice.

A further innovation in GRS is the use of a Full-Height Rigid (FHR) facing (adopted as standard technology for vulnerable lifeline assets such as high-speed railways in Japan), quite
different to the typically utilised (at least in New Zealand) flexible wrap-around, or segmental-block faced GRS systems. New Zealand has a similar seismic risk, yet to date, the use of FHR for vulnerable roadways and the recently nationalised railways, has been limited or non-existent. Thus the use of FHR facing panels for increased seismic performance was investigated in the test series.

**Experimental Study**

The experimental study consisted of a series of seven reduced-scale GRS model walls under seismic excitation conducted using the University of Canterbury shake-table. The models were 900 mm high, reinforced by five layers of stiff Microgrid geosynthetic reinforcement, and were founded on a rigid foundation. The models were faced by a FHR panel connected rigidly to the reinforcement. The soil deposit backfill was constructed of dry dense (target $D_r = 90\%$) Albany sand, compacted by vibration of each layer by the shake-table with a weighted steel plate placed on top of the layer. The influence of the L/H ratio and wall inclination on seismic performance was investigated by varying these important design parameters throughout the testing programme. The L/H ratio tested ranged from 0.6 – 0.9, and the walls were primarily vertical except for one test inclined at 70° to the horizontal.

During testing, measurements of the facing displacement and the acceleration within the backfill were recorded at varying levels of shaking intensity. Mechanisms of deformation, in particular, were of interest in this study. Global and local deformations within the backfill were investigated using two methods. The first used coloured horizontal and vertical sand markers placed within the backfill. The second utilised high-speed camera imaging for subsequent analysis using Geotechnical Particle Image Velocimetry (GeoPIV) software. GeoPIV enabled shear strains to be identified within the soil at far smaller levels than that rendered visible by eye using the coloured sand markers. The complementary methods allowed the complete spatial and temporal development of deformation within the backfill to be visualised.

**Key Findings**

All models displayed a critical acceleration, below which deformations were minor, and above which ultimate failure occurs. Failure was predominantly by overturning, with some small sliding component. During failure, the rate of sliding increased significantly.

An increase in the L/H ratio from 0.6 to 0.9 increased the stability of the wall and decreased deformations accrued prior to failure. Accelerations at failure also increased, from 0.5g to 0.7g, respectively. A similar trend of increased seismic performance was observed for the wall inclined at 70° to the horizontal.
Overturning was accompanied by the development of inclined shear surfaces which started at the wall crest and extended down to the back of the reinforced soil block. With increasing acceleration amplitude, further inclined shear surfaces developed progressively deeper within the retained backfill. Additional shear surfaces also propagated near-vertically up the back of the reinforced soil block, and horizontal failure surfaces propagated from the reinforcement tail, along the reinforcement layer and/or into the reinforced soil block. The initiation of the surfaces were observed to occur at much lower shaking amplitudes (0.1g-0.2g) than is considered in design, highlighting the ductile nature of GRS walls. Failure of the model was observed when an inclined failure surface developed from the wall crest downwards towards the back of the reinforced soil block and intersected with the lowest layer of reinforcement.

For all tests, the reinforced soil block was observed to demonstrate non-rigid behaviour, with simple shearing along horizontal planes as well as strain localisations at the reinforcement or within the back of the reinforced soil block. This observation is contrary to design, which assumes the reinforced soil block zone to behave rigidly.

**Future Research**

The study examined the influence of the L/H ratio and wall inclination on seismic performance, but further work is required in order to determine in detail the existence of an ‘optimum’ L/H ratio, and more details into the effect of wall inclination on behaviour. This study is the first of a research programme which investigates the seismic performance of GRS walls, and determined the methodology to enable future studies to possibly investigate, among other things, the use of local soils as backfill, the addition of a surcharge load, facing connections, and their impacts on seismic performance.
Plain English Summary

Reinforcement of soil enables a soil slope or wall to be retained at steeper angles than the soil on its own could manage. Geosynthetic Reinforced Soil (GRS) wall structures incorporate geosynthetic reinforcement (a strong and flexible engineering plastic grid) layered horizontally into soil walls to stabilise and/or retain the soil. They take less time to construct at a reduced cost (with savings of up to 50%) and perform better in earthquakes than their conventional retaining wall counterparts such as large concrete based gravity and cantilever type retaining walls. Further, GRS walls tend to be more environmentally sustainable, with reduced carbon emissions and embodied energy.

In countries like Japan which has a high earthquake risk, GRS technology is used for important and vulnerable lifeline assets such as high-speed railways. New Zealand has a similar earthquake risk, yet the use of GRS technology has been limited. To help increase the uptake of GRS technology on New Zealand’s roads and railways, a better understanding of GRS behaviour in the New Zealand earthquake environment is required.

To investigate GRS behaviour under earthquake loading, a series of seven reduced-scale GRS model walls were subjected to earthquake shaking on the University of Canterbury shake-table. The models were 0.9 m high, 2.4 m long, and 0.8 m wide, and comprised dry dense sand, reinforced by five layers of stiff geosynthetic reinforcement and rigid front face. Scaling laws applied to seismic shaking were employed so that the wall effectively represented a prototype 4.5m high wall. The length of reinforcement, and the slope of the wall face was varied to investigate these parameters’ influence on wall stability. Each model was shaken with increasing intensity until failure occurred. During testing, acceleration and displacement measurements, as well as three high-speed cameras were used to plot the progression of how the model deformed. This deformation generally occurred with the wall collapsing over its base and sliding forward. The results of the study demonstrated that GRS walls built with longer geosynthetic reinforcement and shallower slopes were more stable and able to withstand stronger earthquakes.

Finally, advanced image analysis software was used to show deformation previously undetectable by eye at relatively low amplitudes of shaking. These results highlight the relative ductility of GRS walls, and have implications to the way GRS systems are designed.
AN INVESTIGATION INTO THE
DEFORMATION BEHAVIOUR
OF
GEOSYNTHETIC REINFORCED SOIL WALLS
UNDER SEISMIC LOADING

A thesis submitted in partial fulfilment
of the requirements for the Degree of

Master of Engineering

at the

University of Canterbury

By Perry Jackson

2010
ABSTRACT

Reinforcement of soil enables a soil slope or wall to be retained at angles steeper than the soil material's angle of repose. Geosynthetic Reinforced Soil (GRS) systems enable shortened construction time, lower cost, increased seismic performance and potentially improve aesthetic benefits over their conventional retaining wall counterparts such as gravity and cantilever type retaining walls. Experience in previous earthquakes such as Northridge (1994), Kobe (1995), and Ji-Ji (1999) indicate good performance of reinforced soil retaining walls under high seismic loads. However, this good performance is not necessarily due to advanced understanding of their behaviour, rather this highlights the inherent stability of reinforced soil against high seismic loads and conservatism in static design practices.

This is an experimental study on a series of seven reduced-scale GRS model walls with FHR facing under seismic excitation conducted using a shake-table. The models were 900 mm high, reinforced by five layers of stiff Microgrid reinforcement, and were founded on a rigid foundation. The soil deposit backfill was constructed of dry dense Albany sand, compacted by vibration (average \( D_r = 90\% \)). The influence of the \( L/H \) ratio and wall inclination on seismic performance was investigated by varying these important design parameters throughout the testing programme. The \( L/H \) ratio ranged from 0.6 – 0.9, and the walls were primarily vertical except for one test inclined at 70° to the horizontal.

During testing, facing displacements and accelerations within the backfill were recorded at varying levels of shaking intensity. Mechanisms of deformation, in particular, were of interest in this study. Global and local deformations within the backfill were investigated using two methods. The first utilised coloured horizontal and vertical sand markers placed within the backfill. The second utilised high-speed camera imaging for subsequent analysis using Geotechnical Particle Image Velocimetry (GeoPIV) software. GeoPIV enabled shear strains to be identified within the soil at far smaller strain levels than that rendered visible by eye using the coloured sand markers. The complementary methods allowed the complete spatial and temporal development of deformation within the backfill to be visualised.
Failure was predominantly by overturning, with some small sliding component. All models displayed a characteristic bi-linear displacement-acceleration curve, with the existence of a critical acceleration, below which deformations were minor, and above which ultimate failure occurs. During failure, the rate of sliding increased significantly.

An increase in the L/H ratio from 0.6 to 0.9 caused the displacement-acceleration curve to be shallower, and hence the wall to deform less at low levels of acceleration. Accelerations at failure also increased, from 0.5g to 0.7g, respectively. A similar trend of increased seismic performance was observed for the wall inclined at 70° to the horizontal, when compared to the other vertical walls.

Overturning was accompanied by the progressive development of multiple inclined shear surfaces from the wall crest to the back of the reinforced soil block. Failure of the models occurred when an inclined failure surface developed from the lowest layer of reinforcement to the wall crest. Deformations largely confirmed the two-wedge failure mechanism proposed by Horii et al. (2004).

For all tests, the reinforced soil block was observed to demonstrate non-rigid behaviour, with simple shearing along horizontal planes as well as strain localisations at the reinforcement or within the back of the reinforced soil block. This observation is contrary to design, which assumes the reinforced soil block to behave rigidly.
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Perry Jackson

Christchurch, New Zealand
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CHAPTER 1
INTRODUCTION

1.1 Introduction

Reinforcement of soil enables a soil slope or wall to be retained at angles steeper than the soil material’s angle of repose. Systems comprising soil and reinforcement are collectively termed “reinforced soil” and involve the incorporation of reinforcement layered horizontally into select engineering backfill for the purposes or slope stabilisation and/or retaining slopes or walls.

Reinforced soil systems enable shortened construction time, lower cost, increase seismic performance and potentially improve aesthetic benefits over their conventional retaining wall counterparts such as gravity and cantilever type retaining walls (see Fairless 1989; FHWA 2001; Murabsev 2003; El-Emam and Bathurst 2004 as examples). Further, soil reinforcement meets many of the goals associated with sustainable development such as reduced carbon emissions and embodied energy, in addition to cost reductions noted when compared to conventional type retaining walls (Jones 1996; Tatsuoka 2008).

The concept of Reinforced Earth (RE) was first introduced in 1966 by the French engineer Henri Vidal, with the inclusion of steel reinforcing strips into engineering backfill, and connecting these to a stiff concrete facing. Geosynthetic reinforcement was first used in 1971. Both RE and geosynthetic reinforcement are part of a large family of reinforced soil systems that comprise different reinforcement, facing products and design methods.
Geosynthetic reinforcement, with a Full-Height Rigid (FHR) facing panel is the focus of this thesis.

This is an experimental study on the seismic performance of Geosynthetic Reinforced Soil (GRS) retaining walls. The study consisted of a series of seven reduced-scale model tests conducted on the University of Canterbury shake-table. Two key parameters that influence seismic behaviour, namely the reinforcement length to height ratio, and the wall inclination, were systematically varied during testing. Facing displacements, accelerations within the backfill, and backfill deformation was measured during each test.

Mechanisms of deformation, in particular, were of interest in this study. Global and local deformations within the backfill were investigated using a combination of sand markers, and high-speed camera imaging and subsequent analysis using Geotechnical Particle Image Velocimetry (GeoPIV) software. GeoPIV allowed a more in-depth picture of the strain field in selected regions of the backfill to be determined (White et al. 2003).

This Chapter will first briefly introduce the concept of GRS retaining walls and the basis of their design. A general overview of GRS retaining wall performance during previous earthquakes in the United States, Japan and Taiwan is provided in Section 1.3. Their recorded performance has in general, been described as “excellent” (Sandri 1997), and provides further impetus for their increased use in New Zealand, a country of similarly significant seismic risk.

The New Zealand context is discussed in Section 1.4 and first provides a brief overview of GRS use in New Zealand. An argument advocating the use of Full-Height Rigid (FHR) facing panels in New Zealand is presented; as FHR is the preferred facing method in Japan for high-speed rail that demands high rigidity and seismic resistance. The benefits to sustainable development associated with GRS are discussed subsequently.

The objectives of this research project, the experimental scope and the organisation of this thesis is provided in Section 1.5.
1.2 Geosynthetic-Reinforced Soil retaining walls

1.2.1 General characteristics of GRS retaining walls

Geosynthetic-Reinforced Soil (GRS) is a derivative of the original reinforced earth concept developed in 1966. The original reinforcement comprised steel strips layered into the retained soil; this system is now trademarked Reinforced Earth®. Different reinforcement materials, design methodologies and construction techniques have since been derived from the original Reinforced Earth concept; however the fundamental engineering principles of soil reinforcement are the same. The use of geosynthetic reinforcement, a polymeric material, is the subject of this study.

A reinforced soil wall and a conventional cantilever retaining wall are compared in Figure 1-1 below. The following key features of the reinforced soil wall are of note: a reinforced soil block comprised of reinforcement layered into select cohesionless granular fill, a facing type (which can vary with design), a connection between the reinforcement and facing, a retained backfill behind the reinforced soil block, and a reduction in the concrete and foundations necessary as compared to the conventional cantilever retaining wall (RW).

![Conventional cantilever type and alternative reinforced soil retaining wall](image)

Figure 1-1. Conventional cantilever type retaining wall (a) and the alternative reinforced soil retaining wall (b) with reinforced zone, facing and connection (between reinforcement and facing) highlighted. Redrawn from FHWA, 2001.

The important components of a GRS retaining wall are described below:

**Geosynthetic-reinforcement** — A polymeric material comprised either of polyester, polyethylene or polypropylene which is manufactured into geotextile (sheets) or geogrid (planar grid-like arrangement). The reinforcement provides tensile capacity to the soil.
Engineering backfill – Generally cohesionless, free-draining granular fill. This ensures a good frictional interaction with the reinforcement and prevents the build-up of pore water pressures.

Facing – A number of facing types exist for GRS structures, based on aesthetic requirements, proprietary aspects, and structural design considerations. Two types are commonly used in New Zealand: Wrap-around facing whereby the layered geosynthetic is looped around the wall face; and Segmental Retaining Wall (SRW) facing where discrete concrete panels or brick units are stacked and connected to the reinforcement with lips, shear pins and/or friction.

The method of Full-Height Rigid (FHR) panel facings as used in Japan (Tatsuoka 2008) is discussed in Section 1.4.2 below and is used in this study.

Connections – Specifies the method used to connect the reinforcement layered into the fill with the facing system. The connection can be either rigid as in the case of mechanical connection to a concrete panel, or non-rigid as in the case of a SRW, which relies on friction generated between the concrete facing panels and reinforcement.

1.2.2 Design of Geosynthetic-Reinforced Soil

Current design techniques for GRS walls utilise either limit equilibrium or limit state approaches. There are a number of different methods and codes available. For instance, the British Standard (BS 8006:1995) (1995) specifies a limit state design, while the FHWA (2001) specifies a limit equilibrium approach. The New Zealand design guidelines prepared by Murashev (2003) adopt a limit state approach.

Aside from differing design philosophies; the underlying concept of geosynthetic reinforced soils is simple: geosynthetic reinforcement inclusion provides tensile capacity to the soil and allows slopes and walls to be constructed at angles steeper than the soil material’s angle of repose.

Figure 1-2 below shows the reinforcing mechanism. It can be seen that the reinforcement resists the formation of a potential failure surface within the reinforced soil block and is anchored in the resistant zone. The ‘pullout’ capacity of the reinforcement is hence determined by the length of reinforcement, L_{e}, within the resistant zone, and the interaction between the soil and reinforcement.
Design separates possible failure modes into external stability of the reinforced block as a rigid composite mass, and the internal stability of the reinforced block. Modes of failure considered in an external stability analysis are shown in Figure 1-3 and include: (a) Sliding, (b) Overturning, (c) Bearing Capacity Failure, and (d) Deep-Seated Failure.
Figure 1-4: Internal modes of failure. From Murahsev (2003).

Internal stability analysis is concerned with the integrity of the reinforced soil block. This can lead to the failure modes shown in Figure 1-4 and includes: (a) Reinforcement pullout, (b) Reinforcement rupture, and (c) Internal sliding of layers upon one another.

It is obvious that three key design parameters of the reinforced soil block are: reinforcement layout which concerns the vertical spacing between reinforcement layers ($S_v$), and length-to-wall-height ratio ($L/H$); and the inclination of the wall. These design parameters affect wall stability under self-weight, external and seismic loading.

In general static design, different design methods will predict generally similar vertical spacing and length of reinforcement requirements. However in the case of seismic design, different design methods can predict very different vertical spacing and length of reinforcement, indicating that the behaviour of GRS structures under seismic loading is still not fully understood.
1.2.3 Seismic design

There is no widely held consensus on seismic design procedures (Murashev 2003), and many design codes (for instance BS 8006:1995) do not include seismic design at all. Despite this, GRS walls have performed very well in recent earthquakes and this is discussed in Section 1.3 below. Hence the behaviour of GRS walls during earthquakes requires further clarification (Watanabe et al. 2003). The current knowledge of the GRS behaviour under seismic excitation and the seismic aspects of design are discussed in Chapter 2.

1.3 Performance during some recent earthquakes

There is considerable evidence to highlight the good performance of reinforced soil retaining walls during recent earthquakes such as that in Northridge (1994), Kobe (1995), and Chi-Chi (1999). During these earthquakes, significant damage of conventional retaining wall structures was reported, whilst reinforced soil structures demonstrated limited to no damage (Ling et al. 2001; Sandri 1997; Tatsuoka et al. 1996). In many cases, the reinforced soil walls either had no seismic design, or seismic design which considered a Peak Ground Acceleration (PGA) of only 20% of that actually experienced (Gazetas et al. 2004). This highlights the inherent stability of reinforced soil against high seismic loads and conservatism in current design practices (White and Holtz 1997).

The field experiences of the abovementioned earthquakes are summarised briefly below.

Northridge Earthquake (1994)

The 1994 Northridge earthquake of California, United States, was of moment magnitude 6.7, with the duration of strong shaking lasting 10 to 15 seconds. Vertical shaking components up to 1.5 times higher than horizontal accelerations were recorded by Shpakal et al. (1994), as reported by Sandri (1997), and this was a large factor contributing to the earthquake’s relative destructiveness.

Sandri (1997) conducted a review of known geogrid-reinforced Segmental Retaining Walls (SRW) over 4.6 m high. All 11 GRS walls assessed by Sandri (1997) post-earthquake were in “excellent” condition. In particular, the Valencia water treatment plant GRS wall, 6.4 m high and approximately 8 km long, was subjected to peak horizontal accelerations of 0.5g, however the design considered only a peak horizontal acceleration of 0.3g (Cai and Bathurst
Post-earthquake inspection by Sandri (1997) revealed no residual displacement between the segmental block face and soil, indicating a good connection between reinforcement and the facing blocks. Some intermittent surface tension cracks (< 6 mm wide) were noted near the back of the reinforced soil block, and this would indicate mass movement of similar magnitude. In comparison, two cantilever walls located close by experienced significant residual deformation, and required extensive repairs.

Kobe Earthquake (1995)

The Kobe earthquake of Japan in 1995 was magnitude 7.2 and caused widespread structural damage (Tatsuoka 2008). Peak ground accelerations of around 0.5g were recorded (Koseki et al. 2006).

Tatsuoka (2008) reports a number of old gravity, leaning and masonry type retaining walls which demonstrated complete collapse, while a number of modern reinforced concrete cantilever walls were also seriously damaged. In contrast, a number of GRS walls with Full-Height Rigid (FHR) facings, subjected to similar severe ground motions, performed very well.

One GRS wall with FHR facing called the Tanata wall, supported a railway line and was investigated in detail by Tatsuoka (2008). The 6.2 m high Tanata wall was subjected to an estimated PGA of 0.7g and survived the earthquake with limited deformation that included tilting of the wall of 26 cm and sliding at the base of 10 cm. Tatsuoka (2008) notes that the Tanata wall did not have extended top layers of reinforcement as was the case for the other walls which performed better than the Tanata wall.

Next to the GRS wall was a reinforced concrete cantilever retaining wall supported by a row of large-diameter bored piles (even though the subsoil conditions were the same as for the GRS wall). This wall demonstrated similar displacements as the GRS wall, however cost approximately double to triple that of the GRS wall due to the bored-pile foundations required. Hence, the relative performance of the GRS wall was considered satisfactory.

Given the good performance of GRS walls, reconstruction efforts of a number of conventional retaining walls which failed during the Kobe earthquake have since focussed on the use of GRS walls with a FHR facing.
Chi-Chi Earthquake (1999)

The Chi-Chi earthquake of Taiwan in 1999 was a 7.3 magnitude earthquake that resulted in 2200 people being killed and extensive structural damage. PGA's larger than 1.0g were recorded. Significant vertical accelerations was also a feature of the ground motion (Ling et al. 2001).

The geography, population density and construction and design methods utilized make GRS application in Taiwan unique. For instance, because of the hilly terrain and high land costs, wall heights of up to 40 m are quite common with the use of in-situ soils in place of select granular backfill. Because of the proprietary nature of the wall systems employed, the majority of walls were faced with modular blocks in a SRW fashion (Ling et al. 2001).

Design of GRS walls is often carried out by geosynthetic reinforcement manufacturers rather than geotechnical specialists. Thus the design risk is carried by the manufacturers rather than geotechnical specialists, and may result in conservative design for static conditions and higher costs. However, there is often limited attention paid to seismic design aspects.

Ling et al. (2001) conducted a post earthquake review of some of the GRS structures around the central Taiwan region. In general, Ling et al. (2001) found many failure cases of stone, reinforced concrete, and tie-back retaining walls. Additionally, a number of geogrid reinforced SRW were also found to have failed. These walls failed primarily with deformation of the modular block facing via sliding, toppling or local instability with bulging evident near the base of the structures. These deformation patterns demonstrated the importance of both seismic design and good connections between the facing blocks and the reinforcement.

In one example, a SRW approximately 10 m high in the Chung Hsin New Village consisted of both reinforced and un-reinforced sections and provided a good comparison of their relative performance. The un-reinforced section had collapsed, whereas the reinforced wall remained stable. This demonstrated the earthquake resistance of the reinforced soil wall.

1.4 New Zealand context

1.4.1 New Zealand use of GRS
Design of GRS in New Zealand is made using several different overseas standards and
design guidelines i.e. Federal Highways Administration (FHWA 2001), British Standards
Institute (BS8006:1995), Australian Standard (AS4678-2002), and the Deutsches Institut
Bautechnik (DIBt). Manufacturers of reinforcement such as Tensar and Stratagrid, also
produce design methods and guidelines. Hence, GRS structures in New Zealand have been
built with varying resistance to static and seismic loads, and thus different seismic risk (Murashev 2003).

Because of this uncertainty in design, and because the use of GRS in New Zealand was
increasing, Transfund New Zealand (now part of the New Zealand Transport Agency)
commissioned ‘Guidelines for Design and Construction of Geosynthetic-Reinforced Soil
Structures in New Zealand’ prepared by Murashev (2003). The Guidelines are based on a
limit state approach and combine the BS 8006:1995 and FHWA (2001) standards, as well as
research to formulate a New Zealand approach to GRS design.

Murashev (1998) also conducted a survey of all GRS structures constructed in New Zealand
up to 1998. The survey found at least 54 GRS structures that had been completed. The
structures ranged in wall height from 2 m to 13 m. Roughly an equal number of walls
(defined as structures inclined at angles larger than 70° to the horizontal) and slopes (inclined
at angles less than 70° to the horizontal) were surveyed.

Of the structures surveyed, nearly all were for road applications including slip rehabilitation,
embankments, and slope stabilisation. The majority of these roads were minor State
Highways or rural roads. Only 4 structures were recorded as for private purposes and
included support for a carparking facility, and landscaping.

The predominant method of facing for both slopes and walls was wrap-around methods, and
this reflects the low structural performance necessary for the mostly rural applications.
Around 30% of the walls were faced by modular keystone blocks or precast concrete blocks.

All of the walls and slopes surveyed by Murashev (1998) were reported to have performed
satisfactorily under static conditions. As yet however, no GRS structure in New Zealand has
undergone significant seismic excitation, and therefore there is little local experience of their
seismic performance. Hence the field experiences of the abovementioned earthquakes can be
generalised and applied to the New Zealand context.
The case histories above demonstrate that there has been some cases of large deformation and/or failure of GRS Segmental Retaining Walls (SRW). It should be restated that segmental blocks, and wrap-around geogrids, are the predominant facing types of existing GRS structures in New Zealand (up to 1998, at least). Hence, a similar performance as that which occurred in the Northridge and Chi-Chi earthquakes could be expected.

In contrast, Japan has adopted the use of GRS walls with a Full-Height Rigid facing as standard technology for vulnerable lifeline assets such as high-speed railways. New Zealand has a similar seismic risk, yet to date, the use of FHR facings for vulnerable roadways and the recently nationalised railways, has been limited or non-existent. Thus the use of FHR facing panels, as opposed to the more typical (at least in New Zealand) SRW and wrap-around faced GRS walls, is the facing method used for model tests in this research.

1.4.2 Full-Height Rigid facing

After the good performance of GRS walls with a Full-Height Rigid (FHR) panel facing during the 1995 Kobe earthquake, Tatsuoka (2008) reports that reconstruction of failed conventional type retaining walls utilised the GRS FHR technology. Because New Zealand has a similarly high seismic risk, it is argued that this technology could also be used in New Zealand. Tatsuoka (2008) provides details of the construction and additional stability achieved during seismic events as compared to conventional retaining walls.

The staged-construction procedure of GRS FHR walls is described in the following steps:

1. A small foundation is constructed for the FHR facing.

2. The geosynthetic reinforced wall is faced with gravel-filled gabions wrapped-around by reinforcement and is compacted in layers as is typical of standard construction for wrap-around faced GRS walls.

The gravel-filled gabions provide three services: a) They create a temporary facing structure for good backfill compaction; b) they act as a drainage layer directly behind the wall face; and, c) they protect the geosynthetic reinforcement and FHR panel connection from relative displacement of the reinforcement and backfill soil post-completion of the wall.
3. At completion of the wrap-around GRS wall, the wall is left to consolidate (if at all) under static conditions.

4. Finally, a thin and lightly steel-reinforced concrete facing is cast in place once ultimate deformation is complete. This creates a single rigid facing panel and a good connection with the exposed geosynthetic reinforcement wrapped around the gravel-filled gabion baskets.

It is noted that there is no need for an external propping support as in the case of conventional reinforced concrete retaining walls. This is especially advantageous in sites with restricted access or difficult terrain.

Tatsuoka (2008) explains a number of advantageous features, as compared with a conventional reinforced concrete retaining wall, that the completed GRS wall with FHR facing demonstrates. These are summarised as follows:

*Activation of high earth pressure at the rigid connection with the FHR facing*

A rigid connection ensures that high earth pressures and thus high tensile stresses can be activated in the reinforcement. This condition generates high lateral confining pressures and hence high strength of the reinforced soil.

*Full-Height Rigid facing panel*

The FHR facing panel has a high structural integrity which acts to resist localised failures of the wall face (failures such as those demonstrated by GRS SRW faced structures in the Chi-Chi earthquake of 1999). Instead, the location of the critical failure surface is forced to intersect with the FHR toe, resulting in greater resistance against earthquake loading. Additionally, three-dimensional effects make the wall very stable against concentrated vertical and lateral loads.

*Geosynthetic reinforcement*

The geosynthetic reinforcement acts to support the FHR facing panel at regular intervals and reduces the overturning moments and sliding forces acting at the facing toe. Hence there is a reduced need for foundations than compared with a conventional cantilever reinforcement concrete structure, whereby the sole resistance to sliding and overturning is activated at the wall toe. In these cases there is often the need for pile foundations.
The above mentioned advantages of FHR faced GRS walls over SRW GRS and wrap-around faced walls are evidenced by increased performance during previous earthquakes and model tests as reported by Matsuo et al. (1998).

1.4.3 Sustainable development agenda

Definitions of sustainable development in New Zealand often converge on the ‘Brundtland’ definition. For engineering purposes, this relates to the equitable use of economic, social and environmental resources between current and future generations (Statistics New Zealand 2009). Further, the future introduction of a price on carbon will likely have an impact upon private and public decision-making and the selection of one particular retaining wall system over another.

In this context, GRS wall systems are compared with conventional reinforced concrete retaining wall systems along economic, environmental and social criteria to determine its potential in achieving national and international sustainable development goals.

Economic benefits

A number of advantages of reinforced soil structures over conventional reinforced-concrete (RC) cantilever retaining walls (RW) include: Rapid construction without the need for large construction equipment and many experienced labourers, reduction in space required/land acquisition, reduced need for rigid/deep foundations (FHWA 2001). These advantages lead to reported cost reduction of up to 75% in the UK (Jones 1996) and 50% in the USA compared to RC cantilever RWs (Koerner et al. 1998) as reported by Koseki et al. (2006). In Japan, Tatsuoka (2008) also comments on the cost-effectiveness of GRS FHR walls as compared to RC cantilever RWs.

Environmental benefits

Jones (1996) conducted an ecological audit of GRS systems to remove possible commercial distortions in the above described economic cost of GRS systems. The audit was made with reference to a RC cantilever RW system over the life-cycle of both structures. For ecological parameters such as energy, labour, dust and sulphur dioxide emissions, and despoiling of land, the GRS system is ecologically cheaper by roughly 30% than RC cantilever RWs. However the GRS system requires roughly 20% more process water than the compared RC cantilever RW.
Further, Tatsuoka (2008) notes that the use of a GRS FHR system results in a reduction in total CO₂ emissions compared to conventional retaining technology. Thus in addition to the environmental benefits, the cost on carbon is likely to also result in further cost reductions for a GRS wall, when compared to conventional RC RWs.

Social benefits

It is unlikely there is any particular substantive social benefit in the trade-off between GRS vs conventional RWs systems as defined by the Statistics New Zealand social dimension definition (2009). However, as noted above, the need for experienced labour and construction machinery is reduced (FHWA 2001), and this could be an advantage in developing countries where such skills and equipment is in short supply.

From the above arguments it can be seen that GRS systems offer substantial benefits in meeting sustainable development goals and that as part of this, substantial benefits in cost effectiveness over conventional RW systems. However Murashev (2003) notes that unless GRS systems in New Zealand are fully understood, and the uncertainty in design is removed, consultants will prefer to use conventional structures. Further, if a GRS design is selected, consultants often impose strict performance-based specifications upon contractors to carry all design and construction risks. This results in higher costs to Road Controlling Authorities in New Zealand than could otherwise be achieved.

1.5 Research objectives

1.5.1 Research aims

This study focuses on the seismic performance of GRS retaining walls, particularly pre-failure deformation. The influence of reinforcement length-to-height ratio (L/H) and the inclination of the wall on this seismic performance is investigated at varying levels of shaking intensity. The specific objectives of the project were to:

- Develop procedures for shake-table tests on GRS walls.
- Quantify the influence of the L/H ratio and wall inclination on seismic behaviour.
- Identify failure mechanisms and patterns of deformation. Utilise GeoPIV to examine in detail deformation within the wall during seismic loading.
• Identify critical issues for further research studies.

To achieve the abovementioned research aims, a series of 7 reduced-scale model tests were conducted using the University of Canterbury shake-table. The L/H ratio and wall inclination was varied from test to test and wall facing displacement, acceleration within the backfill, and deformations within the backfill were measured during testing. Details of the experimental model are provided in Section 3.

1.5.2 Organisation of Thesis

This thesis consists of six main chapters.

Chapter 2 examines the additional aspects considered in a seismic design, which consider the inertia force, seismic earth pressure, acceleration amplification and the critical acceleration threshold of the structure. The static and seismic performance of GRS walls under field and model scale conditions is discussed. Chapter 3 provides the details of the experimental model and in particular provides details of the model similitude such that results at model scale can be generalised at prototype scale.

Chapter 4 presents the main body of results from the shake-table tests. It is divided into 2 parts. The first part presents the raw results from one of the tests, Test-6, followed by analysis of these results using established techniques. The second part presents the raw results from the entire tests series and conducts parametric analysis to determine the influence of the L/H ratio and wall inclination on seismic performance.

The deformation data obtained from the test series is presented in Chapter 5 in two different forms. The first method uses sand markers within the backfill to plot the progression of deformation throughout the wall during testing and enable final deformation patterns to be viewed by eye. The second method utilises GeoPIV to more accurately determine and quantify the deformation in detail, in particular, at early stages in its development, when deformation was not visible to the naked eye.

Chapter 6 concludes the study and makes some recommendations for design and future research.
References


CHAPTER 2
LITERATURE REVIEW

2.1 Introduction

A review of current static design methodology is not the purpose of this literature review. However, such a review may be found in the New Zealand Guidelines prepared by Murashev (2003). Rather, the extra design considerations for GRS walls under seismic loading, and their treatment, are discussed in Section 2.2. Specifically, this includes inertial and earth pressure forces active on the GRS structure during an earthquake. The impact of acceleration amplification on these additional seismic forces is explained.

The critical acceleration of a structure is discussed as a threshold acceleration above which, theoretically, permanent deformation occurs. Hence, the structure-specific critical acceleration provides an important measure of stability and is a key parameter in performance-based design approaches.

Significant research has been conducted to support key design assumptions and determine behaviour under seismic loading. However, Koseki et al. (2006) states the need for experimental research that further clarifies:

- Methods to evaluate earth pressures under high seismic loads
• Methods to evaluate displacements and/or deformation of earth structures under high seismic loads

In light of these needs, Section 2.3 discusses research relating to the reinforcement length-to-wall-height ratio (L/H) and wall inclination on stability and seismic performance. Section 2.4 looks at the mechanisms of deformation of GRS walls and how these contribute to their seismic performance.

2.2 Seismic design aspects

2.2.1 General

Static design methods produce GRS wall designs that perform well under seismic loading conditions, though often without any form of seismic design (Gazetas et al. 2004; Ling et al. 2001). This implies a high degree of conservatism in current static design methods, and underlines a lack of understanding of GRS behaviour under high seismic loading (Koseki et al. 2006).

Seismic design is considered in codes/guidelines such as FHWA (2001) and Murashev (2003) and is based mostly on pseudo-static methods. However different methodologies still predict different reinforcement densities (i.e. vertical spacing and lengths). Additionally, there are further differences in how researchers believe seismic issues should be treated. These differences centre mostly on:

• The use of partial factors vs global factors of safety

• Selection of an appropriate horizontal acceleration coefficient for design and its relationship with a site specific Peak Ground Acceleration (PGA)

• How amplification of design accelerations within the backfill should be treated

• Prediction of stability and deformation.

The first issue is code-dependant, while the second issue is out of scope of this report and a discussion of local site effects and PGA selection can be found in Kramer (1996). The latter two issues are discussed in the following sections.
A number of methods are available which consider additional seismic forces. Shukla (2002) divides seismic analysis procedures into roughly three categories:

- Pseudo-static methods
- Allowable-displacement methods
- Dynamic finite element/finite difference methods.

Pseudo-static methods are based on conventional static limit equilibrium analysis extended to include additional destabilising forces resulting from an earthquake ground motion. Mononobe-Okabe (MO) earth pressure theory is used to determine seismically induced forces (Okabe 1926). This method is specified by Murashev (2003) and FHWA (2001) for the design of GRS structures.

Where large ground motions are predicted, greater than a PGA of 0.3g, large scale deformation and/or sliding could be expected. Design based on Pseudo-static methods using MO theory is often highly uneconomical and impractical (Murashev 2003). In these cases, Murashev (2003) and FHWA (2001) suggest an allowable displacement approach should be used where some deformation would not result in structural failure. This allows the design to be based on a reduced horizontal acceleration coefficient, $k_h$, resulting in a more economical and practical design (Murashev 2003).

Further, the limit equilibrium approach cannot predict deformation or displacements, only the onset of instability. Thus a number of approaches, based on Newmark sliding-block theory (Newmark 1965) exist to predict order-of-magnitude displacement of the structure under earthquake loading. However, these approaches depend on an accurate estimation of critical acceleration (i.e. the acceleration at which the permanent deformation occurs), otherwise it is difficult to predict displacement accurately (Koseki et al. 1998). Researchers' opinions also differ on the best method to estimate a structure’s critical acceleration.

Dynamic finite element/finite difference methods have been reported in the literature for the analysis of GRS structures with seismic loading (El-Emam et al. 2004; Hatami and Bathurst 2000; Ling et al. 2004; Ling et al. 2005; Segrestin and Bastick 1988). However a discussion of these advanced methods is out of scope of this report.
2.2.2 Inertia forces

During an earthquake, horizontal and vertical acceleration components are propagated up through the GRS wall; these accelerations act to increase the earth pressure, and create inertial loads on the facing and active wedge. The additional seismic forces acting on the reinforced soil block are shown in Figure 2-1 below.

![Inertia forces](image)

**Figure 2-1.** Inertia forces and dynamic earth pressure increments additional to static forces acting on a GRS wall during an earthquake. From Murashev (2003).

Figure 2-1 shows the inertia force, and earth pressure distribution acting on the wall. The inertia force is equal to the mass, \( m \), factored by the design seismic acceleration coefficient, \( k_s \), and acceleration due to gravity, \( g \). The inertia forces are shown for the reinforced soil block, \( F_{IR} \), and the inclined backfill, \( F_{IS} \), which contribute to the total inertia force, \( F_{IT} \). Should there be a substantial difference in the weight of the facing type used and the soil, then the facing mass should also be considered to contribute to the total inertia force.

Murashev (2003) specifies that only the mass contained in the front 0.5H of the reinforced soil block is considered to contribute to the inertial force, \( F_{IR} \). This is because the inertia...
forces of the reinforced soil block and retained backfill will be unlikely to reach peak values at the same time.

Figure 2-1 also shows the total earth pressure distribution acting on the back of the wall made up of its static and dynamic seismic earth pressure components. The resultant total dynamic seismic force, \( F_{AE} \), comprises the resultant static force, \( F_s \), and the resultant dynamic increment force, \( \Delta F_{AE} \). The derivation of the resultant earth pressure forces is explained below.

### 2.2.3 Seismic earth pressure

GRS walls are flexible structures (Gazetas et al. 2004) and as such develop minimum active and maximum passive earth pressures (Kramer 1996). The pseudo-static MO approach is used to determine the seismic earth pressure acting to destabilise the wall Murashev (2003). MO theory is an extension of Coulomb theory with the addition of pseudo-static horizontal and vertical accelerations acting upon the Coulomb failure wedge. Thus for the purposes of design, the total dynamic active soil pressure, \( F_{AEh} \), can be expressed as in Equation 2-1 (based on Murashev 2003; Kramer 1996).

\[
F_{AE} = 0.5 \left( 1 - k_v \right) K_{AE} \gamma h^2
\]  \hspace{1cm} (2-1)

Where \( k_v \) is the design vertical seismic coefficient (as a fraction of acceleration due to gravity, \( g \)). \( \gamma \) is the soil density, \( h \) is the wall height, and \( K_{AE} \) is the total earth pressure coefficient expressed as Equation 2-2.

\[
K_{AE} = \cos^2 \left( \phi - \xi - 90 + \theta \right) \cos \xi \cos^2 \left( 90 - \theta \right) \cos \left( \beta + 90 - \theta + \xi \right) \left[ 1 + \frac{\sin(\phi + \beta) \sin(\phi - \xi - \beta)}{\cos(\beta + 90 - \theta + \xi) \cos(\beta - 90 + \theta)} \right] \]  \hspace{1cm} (2-2)

Where \( \beta \) is the backfill slope angle with the horizontal, \( \theta \) is the inclination of the face with the horizontal, \( \phi \) is the soil angle of friction and \( \xi \) is the seismic inertia angle (Shukla 2002) as defined in Equation 2-3. All other angles are shown in Figure 2-1.

\[
\xi = \tan^{-1} \left[ \frac{k_h}{(1 - k_v)} \right]
\]  \hspace{1cm} (2-3)
Where $k_h$ is the horizontal seismic coefficients (as a fraction of acceleration due to gravity, $g$).

As seen in Figure 2-1 above, $F_{AE}$, can be divided into its static, $F_s$, and dynamic increment, $\Delta F_{AE}$, components. The static earth pressure is distributed hydrostatically, thus the resultant force acts at $h/3$. Seed and Whitman (1970) assume the dynamic force increment to act at 0.6$h$. Hence the location of the resultant total dynamic earth force normally acts at around half the wall elevation, or 0.5$h$ (Kramer 1996).

An approximate solution for the critical failure surface was determined by Zarrabi-Kashani (1979) and reported by Kramer (1996) as shown in Equation 2-4a.

$$\alpha_{AE} = \phi - \psi + \tan^{-1}\left[\frac{-\tan(\phi - \psi - \beta) + C_{1E}}{C_{2E}}\right]$$  (2-4a)

Where $\alpha_{AE}$ is the critical failure surface angle with the horizontal plane and $\delta$ is the interface friction angle between soil and wall face. Factors $C_{1E}$ and $C_{2E}$ are defined in Equations 2-4 (b) and (c).

$$C_{1E} = \sqrt{\frac{\tan(\phi - \psi - \beta)}{\tan(\phi - \psi - \beta) + \frac{1}{\tan(\phi - \psi - \theta*)}}\left[1 + \frac{\tan(\delta + \psi + \theta*)}{\tan(\phi - \psi - \theta*)}\right]}$$  (2-4b)

$$C_{1E} = 1 + \left\{\tan(\delta + \psi + \theta*)\left[\frac{1}{\tan(\phi - \psi - \beta) + \frac{1}{\tan(\phi - \psi - \theta*)}}\right]\right\}$$  (2-4c)

It is important to note that the wall face inclination angle, $\theta*$, is made with respect to the vertical, as defined by Kramer (2006).

The inclination of the wedge is shallower than the static case (Kramer 2006), and this reflects a larger active wedge formed due to the introduction of seismic forces.

As MO theory is an extension of Coulomb theory, the limitations associated with Coulomb theory are applicable. One of these is that the reinforced soil block is considered rigid, and by definition, the shear wave velocity of the soil profile is considered infinite, and the base acceleration propagates instantly into the block. The pseudo-static analysis precludes
deformation of the rigid block and the dynamic changes in the peak ground acceleration during wave propagation. It thus becomes difficult to assess the appropriate design seismic coefficient to determine dynamic earth pressures (Kramer 2006).

An additional problem with the MO pseudo-static method is that it has been demonstrated to over-predict earth pressures and it generally results in a highly conservative design (e.g. Kramer 1996; Murashev 2003; Wood 2008). Further, Koseki et al. (1998) conducted reduced-scale model tests on both conventional type and reinforced soil walls, and compared the MO-predicted failure plane angles with those observed at failure. The MO calculated failure planes were shallower than those observed, with the largest discrepancy observed for the reinforced soil wall.

Contrasting with these results are studies conducted by El-Emam and Bathurst (2004) on reduced-scale reinforced soil models which showed the predicted MO failure planes to be steeper than the observed failure plane. The comparison of the calculated failure surface angle, and that interpreted from inclinometer tube measurements is shown in Figure 2-2.

![Image](image_url)

**Figure 2-2.** Failure plane interpreted from inclinometer tubes and extensimeters compared with the predicted failure surface by MO theory. From El-Emam and Bathurst (2004).

The discrepancy between the observed and predicted failure planes made by MO analysis for both studies, in general, shows that the behaviour and deformation of reinforced soil walls is not well understood.
2.2.4 Acceleration amplification

Considerable evidence from previous earthquakes notes local site effects (Kramer 2006). Experimental evidence from reduced-scale model tests on reinforced soil models show that these structures are no exception, and that the ground motion is modified within the structure (El-Emam and Bathurst 2004; Law and Ko 1995; Nova-Roessig and Sitar 2006). Further, De and Zimmie (1998) show the soil-reinforcement interaction to influence ground motion propagation through the structure in reduced-scale model tests in a geotechnical centrifuge.

Whilst an in-depth discussion of the fundamentals of acceleration amplification is not the subject of this chapter, a single degree of freedom (SDOF) model is used to briefly illustrate the problem and soil parameters involved. The SDOF model has a stiffness, $k$, and is undergoing forced vibration by ground motion at frequency, $\omega$. The model has a fundamental circular frequency, $\omega_0$ as shown in Equation 2-6 below, proportional to its stiffness and mass, $m$.

$$\omega_0 = \frac{k}{\sqrt{m}}$$  \hspace{1cm} (2-5)

The second order differential equation describing the motion of the soil system is solved for the soil profile's displacement response as a function of the input ground motion and an amplification factor, $AF$. The amplification factor is dependent on the system damping characteristics and the ratio of the frequency of the input ground motion to the fundamental frequency of the soil structure.

Figure 2-3 plots the amplification factor for the SDOF model as a function of damping ratio and ratio of the excitation frequency to the SDOF natural frequency.
Figure 2-3. Effect of soil properties on the dynamic amplification of acceleration within the reinforced soil wall. (From Law and Ko)

As seen in Figure 2-3, a ‘resonance’ condition occurs when the applied frequency of ground motion is equal to the natural frequency of the SDOF model. This would lead to excessive amplification of motion within the soil profile.

It should be noted that during an earthquake, strain development leads to a reduction in soil stiffness, G, and an increase in damping. Using the SDOF analogy shown in Equation 2-5, this alters the fundamental frequency of the soil profile and soil wall system. Hence the amplification factor will also change during an earthquake.

Amplification of acceleration is a design concern because it can generate larger accelerations leading to larger destabilizing dynamic earth pressure and wall inertia. The Australian Standard (AS4678-2002) notes that acceleration amplification can act to increase seismically induced displacements. However, as noted above in Section 2.2.3, the pseudo-static methods assume that the acceleration is uniform over the height of the structure. Hence to account for amplification of design accelerations in their pseudo-static analyses, FHWA (2001) modify the design horizontal acceleration coefficient, $k_{h,des}$, as in Equation 2-6.

$$ k_{h,des} = (1.45 - k_h) k_h $$  \hspace{1cm} (2-6)

Where $k_h$ is limited to 0.45, whereby after that $k_{h,des}$ is assumed to equal $k_h$. The design acceleration, $k_{h,des}$, is used for both external and internal stability calculations and applied at around mid-height of the wall.
In contrast, where some lateral movement of the wall can be tolerated, Murashev (2003) suggests the design acceleration, \( k_{h,\text{des}} \) can be reduced 40% for external and internal sliding stability analyses, and acceleration amplification is not considered. However, internal stability analysis of reinforcement pullout and rupture does consider acceleration amplification with Equation 2-7.

\[
k_{h,\text{int}} = (1.3 - k_h) k_h
\]  

(2-7)

Again, \( k_{h,\text{int}} \) is applied at around mid-height of the wall.

The effect of Equations 2-6 and 2-7 is an amplified design horizontal acceleration coefficient for seismic coefficients less than 0.45 and 0.3, respectively. The limit of 0.45g is based on some finite element modelling work by Segrestin and Bastick (1988). The authors note that the work is highly idealised and limited. The different limiting coefficient of 0.3 proposed by Murashev (2003) is unexplained.

Experimental and numerical modeling shows that both amplification and de-amplification occurs in GRS walls. Nova-Roessig and Sitar (2006) showed that wall top motion was amplified by up to 2.3 times for accelerations less than 0.15g, and that de-amplification generally occurred for base input accelerations larger than 0.46g. This would seem to validate the Equation 2-6 used by FHWA (2001).

However, other studies show that the amplification can be more significant than suggested by Equations 2-6 and 2-7. Fairless (1989) reported accelerations at the wall top up to three times larger than that at the wall base. El-Emam and Bathurst (2004) showed amplification of the acceleration at the top of the facing panel up to 2.25 times, with accompanied amplification of the backfill soil up to 1.75 for accelerations larger than 0.5g.

Matsuo et al. (1998) separated the amplification of motions into that in the forward and backward directions. The results showed an amplification of forward accelerations for all levels of shaking, however a de-amplification of backward accelerations at accelerations larger than around 0.3g. The authors attributed this to the soil along the sliding surface being unable to transmit the restoring acceleration up into the sliding block.
In addition to the variant amplification factors used by FHWA (2001) and Murashev (2003) for external and internal stability analyses, further debate also centers on how best to take into account acceleration amplification for design purposes. Cai and Bathurst (1996) argue that the location of dynamic earth pressure increment at 60% the wall height from the bottom of the wall may indirectly account for acceleration amplifications.

Steedman and Zeng (1990) modeled the soil profile with MO assumptions, but with a finite shear wave velocity of the soil profile, instead of infinite shear wave velocity, as in a rigid block assumption. Thus the acceleration within the soil varies as a function of depth and soil behavior such as phase change of ground motion and amplification can be accounted for pseudo-dynamically.

2.2.5 Critical acceleration

The critical acceleration is defined by Bracegirdle (1980) as “the horizontal pseudo-static acceleration acting uniformly over the structure to achieve limiting equilibrium.” For each mechanism of failure, considered typically as rotation and sliding of the reinforced soil block, the structure will have an associated critical acceleration at which point the dynamic factor of safety against this failure mechanism will be less than 1, and permanent displacement will be induced.

A general form of the dynamic factor of safety, $FS_d(t)$ is shown in Equation 2-8 below.

$$FS_d(t) = \frac{\text{available resisting force}(t)}{\text{driving force}(t)}$$  \hspace{1cm} (2-8)

Note that $FS_d(t)$ varies during an earthquake as both the driving and resisting forces are functions of the ground motion response and therefore time. For instance, considering limiting equilibrium against sliding, i.e. setting $FS_d(t)$ equal to unity, the critical acceleration which would induce sliding can be determined. Hence, accelerations larger than this value may generate large permanent displacements.

Model studies have demonstrated the existence of threshold critical acceleration values for gravity retaining walls (Lai 1979) and for reinforced soil walls (El-Emam and Bathurst 2004; Koseki et al. 1998; Matsuo et al. 1998; Nova-Roessig and Sitar 2006; Sakaguchi 1996;
Watanabe et al. (2003). In these studies, accelerations larger than some threshold value generate large permanent displacements.

Matsuo et al. (1998) conducted multiple reduced-scale model tests on geosynthetic-reinforced soil walls. Figure 2-4 shows the normalized residual wall top displacement, \( \frac{d_{w}}{H} \) for each wall type with increasing base input acceleration.

![Graph showing the normalized residual wall top displacement](image)

**Figure 2-4.** Normalised residual wall top displacement, \( \frac{d_{w}}{H} \), as a function of increasing base input acceleration for a number of different model wall types. The relationships are approximately bi-linear and demonstrate evidence of model type-specific critical acceleration (From Matsuo et al, 1998).

The residual displacement of the wall top demonstrates a bi-linear trend for each wall type. That is, deformation is small until some threshold base input acceleration, at which point deformation is suddenly increased. The critical acceleration for the model walls is seen to be around 200 \( \text{cm/s}^2 \) to 350 \( \text{cm/s}^2 \) (0.2g to 0.35g).

Critical acceleration is thus an important parameter which can quantify the outset of instability. A high critical acceleration implies a higher acceleration necessary to induce failure, and reduced deformation at low accelerations.

A number of methods exist to estimate the critical acceleration, based generally on Equation 2-8 above. Elms and Richards (1979) extended MO-theory to include wall inertia forces and base friction, to estimate earthquake-induced rigid-block sliding displacements of gravity retaining walls.
In order to account for other possible modes of failure evident for reinforced soil walls, Cai and Bathurst (1996) extended the sliding block model to determine seismically-induced permanent displacement of GRS SRW. In addition to rigid-block sliding, limit equilibrium stability analysis considered two extra potential failure modes for the GRS SRW as: Sliding along individual reinforcement layers ('internal type failure'); and internal sliding between SRW facing units (facing failure). Analytical equations for the critical acceleration for each mode of failure were determined.

The estimation of critical acceleration suffers from the limitations of pseudo-static limit equilibrium analysis. Matsuo et al. (1998) best illustrated this with a calculation of critical acceleration using pseudo-static methods for GRS models of different reinforcement length and vertical spacing and wall inclination. Despite these differences, the model walls’ calculated critical acceleration for overturning varied between 0.45g – 0.55g and for sliding, varied between 0.6g – 0.75g. However, as shown in Figure 2-4, the observed critical acceleration varied between 0.2g – 0.35g.

2.2.6 Performance-based design

Performance-based design allows designers to predict the behaviour of a structure during an earthquake. The design methodology allows designers to first select an allowable displacement, i.e. performance, which can be tolerated during an earthquake. Tolerance of some movement can lead to a reduction in the seismic coefficient used for design and hence create a more cost effective solution (Richards and Elms 1979). For instance, Wood (2008) determined that in areas of the highest seismic hazard in New Zealand, where up to 100 mm movement of the RW can be tolerated, the PGA can be reduced by up to 50%.

To calculate the permanent displacement of the structure during an earthquake, the sliding block theory proposed by Newmark (1965) is used. The reinforced soil block is treated as a rigid-plastic monolithic mass with a critical acceleration calculated as described in Section 2.2.5. An appropriate acceleration time history is applied to the soil block; permanent displacement occurs only when the ground acceleration exceeds the soil block critical acceleration, or the velocity of the soil block is greater than zero. This concept, and the process of double-integration of the acceleration time history is shown graphically in Figure 2-5 below.
Figure 2-5. Permanent displacement calculation using Newmark’s sliding-block method. From Cai and Bathurst (1996)

The predicted displacement can then be compared with that considered tolerable and the design evaluated against the desired performance.

Where ultimate wall failure is by sliding, and hence closest to the original assumptions made by Newmark (1965), reduced-scale tests by Fairless (1989) and Nagel (1985) demonstrate that the Newmark predicted displacement is in good agreement with experimental results.

However, Matsuo et al. (1998) notes that the GRS walls are subjected to continuous deformation (i.e. do not obey rigid-perfectly plastic behaviour), and this results in an underestimation of displacement when using sliding block models. Further, displacement prediction is highly sensitive to the selection of critical acceleration values for each particular mode of failure. It is therefore vulnerable to the issues described in Section 2.2.5.
2.3 Parameters which influence seismic performance

Leshchinsky (2000) postulated that the main objective of design is to produce safe and economical structures. The non-collapse of GRS in recent earthquakes indicates that the first objective has, in the large part, been achieved with modern design techniques. However, GRS walls predominantly fail in a ductile manner, where displacements, not necessarily ultimate failure, are of concern. Hence, the performance of GRS, at an acceptable cost, is the subject of considerable research (Koseki et al., 2006).

Two important parameters which influence ultimate stability and performance of a GRS wall are the reinforcement arrangement, and the wall inclination. For the static case, there are limited examples of fully instrumented GRS structures in the field, thus there is minimal field data that can be used to assess performance and/or critical design assumptions (Fannin and Hermann, 1990; Zornberg and Arriaga, 2003). Even less field data exists under seismic loading and in many cases the soil properties have not been modelled appropriately making analysis difficult (Ling et al. 2004).

Hence the influence of L/H ratio and wall inclination on static and seismic performance has been the subject of some previous experimental research using tilt-table, shake-table and seismic centrifuge tests (El-Emam and Bathurst 2004; Koseki et al. 1998; Sabermahani et al. 2009; Sakaguchi 1996; Watanabe et al. 2003) as discussed in Sections 2.3.1 and 2.3.2 below.

It should be noted that reduced-scale tests have their limitations and the response of the model can be influenced by low confining pressure, boundary effects and improperly scaled reinforcement mechanical properties (Koseki et al. 2006). Thus an important aspect of all model scale experimental tests is the need to ensure model similitude; otherwise the results at model scale are not directly transferable to proto-type scale.

2.3.1 Influence of Length-to-Height Ratio on seismic performance

Section 1.2.2 shows that the L/H ratio is a key design parameter for wall stability. However, seismic design with large seismic coefficients can lead to MO-theory being indeterminate or can result in a large failure wedge. In these cases, design might specify an unreasonably large or impractical L/H ratio to guarantee stability. This implies a greater cost in geosynthetic-
reinforcement; however the cost is minor compared to the additional cost of select granular backfill and excavation required with a larger L/H ratio.

Both Murahsev (2003) and FHWA (2001) indicate that for seismic design, L/H should be limited to L/H ≤ 1.0 or 1.2. Further, for seismic coefficients, k_s, larger than 0.3, and where some movement can be tolerated, the seismic coefficient can be reduced, based on an assessment of the seismic performance of the GRS wall.

As part of wider experimental parametric studies, various researchers have investigated the influence of L/H on static and seismic performance. Two general research objectives were to examine:

- The impact of the L/H ratio on performance during an earthquake

- The determination of optimum L/H ratio for cost-effective design for seismic events

Investigations of particular interest include those by El-Emam and Bathurst (2004, 2005, 2007), Watanabe et al. (2003), Sakaguchi et al. (1996), Sabermahani et al. (2009) and Nova-Roessig and Sitar (2006), and are discussed in some detail below.

El-Emam and Bathurst (2004; 2005; 2007) conducted a series of reduced-scale model tests using a shake-table and a scaled geosynthetic geogrid with sand glued to its surface to ensure good soil-reinforcement interlock. Parameters such as facing toe condition, wall mass and inclination, vertical spacing, reinforcement stiffness, and L/H were investigated. L/H was set to 0.6 and 1.0. The models were subjected to incrementally increasing base shaking and the influence of these parameters on seismic performance is shown in Figure 2-6.
Figure 2-6. Influence of reinforcement properties, including L/H, vertical spacing (S_v) and reinforcement stiffness (Jmn) for GRS models. NB: The model’s toe condition was hinged (i.e. base sliding completely restricted). (From El-Eman and Bathurst 2007).

Figure 2-6 plots residual wall top displacement, ΔX_T, due to increasing base acceleration amplitude. A shallower curve, i.e. smaller displacement for increasing base input acceleration indicates a higher resistance to seismic loading. As seen in Figure 2-6, an increase in L/H from 0.6 to 1.0 (Walls 1 and 3) resulted in a smaller displacement response. Specifically, from a base input acceleration of 0.45g, the wall reinforced at 0.6 had displacement around 40 mm, roughly twice that of the wall reinforced at L/H = 1.0.

Similar increases in performance due to increased L/H have been noted in other shake-table studies. Watanabe et al. (2003) conducted shake-table tests on 3 conventional and 3 reinforced soil retaining walls with a FHR facing at 1:10 scale. The models were 0.5 m high, constructed to a relative density of 90% and had a 1KPa surcharge load applied. The reinforced-soil models were reinforced by a “geogrid” that comprised soldered phosphor and bronze strips. An irregular acceleration time history based on a Kobe (1995) earthquake record with a predominant frequency of 5 Hz was used as the base excitation. The record was scaled to an initial PGA of 100 gal (~ 0.1g), and then increased incrementally by 100 gal.
Watanabe et al. (2003) set L/H to 0.4 and 0.7 using a scaled reinforcement of phosphor bronze strips. The wall reinforced at L/H = 0.7 had reinforcement roughly 80% longer than the other models; however seismic performance was no better. Again, this indicates an optimum value for reinforcement length. In one model, two upper reinforcement layers were increased in length from L/H = 0.6 to 0.9 and 1.6. This was found to be effective at resisting wedge deformation, and the increased length effectively improved the seismic stability of the GRS wall.

Sakaguchi (1996) conducted reduced-scale model tests using both a shake-table and tilt-table with reinforcement lengths set at L/H = 0.33, 0.66, and 1.0. Scaled geosynthetic geogrid reinforcement was used in the model tests. An increase in L/H from 0.66 to 1.0 was found to be less effective at resisting deformation than an increase in L/H from 0.33 to 0.66 and this suggested an optimum L/H value equal to 0.67.

The geotechnical centrifuge may be used to better model prototype stresses in the ground and thus soil behaviour, than model studies using a 1-g shake-table. Nova-Roessig and Sitar (2006) set L/H = 0.7 and 0.9 with scaled reinforcement of a geosynthetic-textile and mesh of wire strips. As in the above shake-table tests, an increase in reinforcement length decreased the residual displacement of the wall during increasing base excitation.

Section 2.2.4 noted that the composite stiffness of the reinforced soil block is an important factor in dynamic wall response. The impact of L/H on the reinforced soil block stiffness, and thus stability has been studied by Sabermahani et al. (2009). A number of reduced-scale model tests were conducted on vertical walls with wrap-around facing, and varying reinforcement type and lengths. The authors defined a new parameter, $G_{\text{global}}$, as the stiffness of the entire wall. $G_{\text{global}}$ is equal to the stiffness of one layer within the wall, normalised by the confining stress at that layer. The parameter was found to be unique for each reinforcement layout used.

$G_{\text{global}}$ was found to be apparently larger for models with a larger L/H ratio. Thus models with a larger L/H demonstrate a stiffer response than shorter reinforced L/H models at low strains. The parameter displayed conventional strain degradation, and at larger strains corresponding to 0.1%, $G_{\text{global}}$ seemed to approach some limit and was unaffected by the L/H ratio.
These investigations among others suggest an optimum reinforcement ratio, $L/H$, at which point any increase, is not met by a proportional increase in performance (Sakaguchi, 1996; Watanabe et al. 2003; Nova-Roessig and Sitar, 2006). However it is difficult to draw this conclusion precisely as reinforcement properties, approaches to similitude, and testing procedures vary greatly between test arrangements.

### 2.3.2 Influence of wall face inclination on seismic performance

Codes define a GRS slope as one that is inclined at less than 70° to the horizontal, and a GRS “wall” as that inclined greater than 70° to the horizontal. The design procedures for each structure type are different, reflecting differences in structural requirements, reinforcement loads and failure modes.

The effect of slope inclination on slope stability is a basic problem; earth pressure theory predicts a decrease in lateral forces on the wall with an increase in inclination from the vertical (El-Emam and Bathurst, 2005). The impact of wall inclination on stability has also been investigated experimentally (El-Emam and Bathurst 2005; Lo Grasso et al. 2005; Matsuo et al. 1998).

One of the parameters varied during the above noted experiments by El-Emam and Bathurst (2005) was wall inclination, and 2 of the 14 models tested were inclined at 80° to the horizontal. The residual wall displacement with increasing base input acceleration is compared for three walls of different facing inclination and mass in Figure 2-7.
Figure 2-7. Lateral displacement measured at wall top, $\Delta X_t$, for three walls of varying facing mass and inclination ($L/H = 0.6$). From El-Emam and Bathurst (2005).

As seen in Figure 2-7, the parameters governing behaviour was the facing thickness and inclination. At all base accelerations, the model inclined at 80° (Wall 12) showed the smallest displacement, and demonstrated the largest critical acceleration. The results indicated that increasing the facing inclination was more effective in increasing wall stability, than increasing facing mass (El-Emam and Bathurst 2005).

Similarly, Matsuo et al. (1998) reported a 10\% increase in the PGA required to generate the same displacement for a wall inclined 79° to the horizontal compared to a vertical wall.

2.4 Mechanisms of deformation

2.4.1 Observed failure modes of GRS

Matsuo et al. (1998) notes that deformation modes due to shaking are dependant on wall facing and inclination. Hence, a number of experimental studies have been undertaken to validate failure mechanisms used in design (El-Emam and Bathurst 2004; Matsuo et al. 1998; Sabermahani et al. 2009; Watanabe et al. 2003).
The modes of deformation observed in reduced-scale model GRS wall shaking table tests conducted by Watanabe et al. (2003) are shown in Figure 2-8. The experimental details of Watanabe et al. (2003) have been described previously in Section 2.3.1.

Figure 2-8. Residual displacement of 6 types of retaining wall, observed after the final shaking step. From Watanabe et al (2003).
It can be seen in Figure 2-8 that the predominant failure type for all six walls was overturning, with some small component of sliding failure. The conventional-type retaining walls demonstrated varying degrees of overturning, sliding and bearing capacity failures. All types of failure were accompanied by multiple failure surfaces formed within the retained backfill. This is discussed in Section 2.4.2.

El-Emam and Bathurst (2007) conducted a number of reduced-scale tests using a shake-table. All models were 1 m high, had a FHR panel facing, and had varied reinforcement arrangement and toe condition (either free-sliding or hinged). Again, overturning was the predominant mode of failure.

Comparison of the failure modes of the reinforced soil walls reveals that despite different L/H ratios and reinforcement layout, the failure comprised similar overturning and sliding components. Thus the overturning mode is significant and cannot be ignored in stability analyses as proposed by Newmark (1965) and Richards and Elms (1979).

2.4.2 Formation of failure surfaces

Figure 2-8 shows that for the tests conducted by Watanabe et al. (2003), the vertical walls exhibited multiple failure planes with the second failure plane forming upon larger amplitude shaking. For the reinforced soil model walls, the failure planes originated at the base of the interface between the reinforced and retained soil zones. No failure planes were observed within the reinforced zone. This pattern of deformation indicates that when the reinforcement is arranged sufficiently it resists the formation of a failure plane within the reinforced soil block.

In one test conducted by El-Emam and Bathurst (2004), the model was toe-hinged and reinforced with 5 layers of length L/H = 1.0. Overturning accompanied the formation of a failure surface which intersected both the reinforced zone and retained backfill (contrary to the above model tests by Watanabe et al., 2003). This deformation is shown above in Figure 2-2.

Sabermahani et al. (2009) tested vertical walls with a wrap-around facing on the shake-table. Overturning failure was accompanied by multiple failure surfaces which formed in the backfill and extended down to the second or third layer (from the base) of reinforcement.
Bulging of the wall face occurred for walls reinforced with reinforcement of very low stiffness and resulted in the largest displacement being recorded around mid-height of the face (and not at the wall top as in the overturning failure). This failure mode was accompanied by an internal slip surface within the reinforced soil block.

These observations highlight the importance of facing type and reinforcement material properties on GRS seismic behaviour. When a wall is faced with an FHR panel, the failure surface formed on ultimate failure is located at the wall heel or back of the reinforced soil block. In contrast, for non-rigid walls, failure surfaces at ultimate failure may not necessarily form at the heel or base of the reinforced soil block (Tatsuoka 2008). Additionally, depending on the reinforcement material properties and/or reinforcement layout, a failure surface may form within the reinforced soil block.

The two-wedge failure mechanism used in current Japanese design practice is shown below in Figure 2-8 and shows a near vertical failure surface and an inclined failure surface which form behind the reinforced soil block. The deformation observed in the Watanabe et al. (2003) models, in general confirmed the use of this mechanism. However, in these models, no failure plane was observed within the reinforced soil block, shown as dashed line A – B in Figure 2-8.

![Two-wedge failure mechanism](Image)

**Figure 2-9.** Two-wedge failure mechanism used in Japanese design. From Tatsuoka (2008).

For the case of an inclined wall, Matsuo et al. (1998) noted in general a more rotational movement with model wall inclined at 77°. This deformation involved the largest horizontal wall face displacement occurring at one third the height of the wall and was accompanied by the formation of a more circular slip surface (than the corresponding base-case vertical wall).
Lo Grasso et al. (2005) observed similar behaviour in reduced-scale shake-table tests on model walls inclined at 70°. Rotational behaviour accompanied the formation of somewhat curved failure surfaces within the backfill, that could be approximated with the two-wedge failure mechanism (Lo Grasso et al. 2005).

However, there are differences in the development of failure between GRS models on the geotechnical centrifuge and shake-table. For instance, a well-defined failure by overturning and sliding (with distinct failure surfaces clearly visible within the backfill) is seen in shake-table models. In contrast, a well-defined failure mechanism is not always visible in model testing using a centrifuge (Howard et al. 1998; Nova-Roessig and Sitar 2006).

Nova-Roessig and Sitar (2006) conducted an experimental study using a centrifuge on slopes inclined at 63° to the horizontal, with wrap-around facing. The model slopes failed in a ‘ductile’ manner with an absence of a defined failure surface within the backfill, and high vertical settlements and outwards slumping of the slope face. Howard et al. (1998) report a centrifuge study on six vertical face model walls faced with discrete panels. For all tests (except one with the shortest reinforcement, L/H = 0.5) a vertical failure surface behind the reinforced soil block was observed. However, for the test reinforced at L/H = 0.5, deformation was accompanied by a defined inclined failure surface and graben wedge behind the reinforced soil block.

The difference in behaviour between the two centrifuge tests, again shows the importance of facing type and model inclination on model response. However, the general absence of inclined failure surfaces observed, indicates that the centrifuge could have influenced model behaviour to occur in a more ‘ductile’ manner as opposed to testing at 1-g on the shake-table. This difference is likely due to higher confining stress levels achieved with a centrifuge.

2.4.3 Rigid-block assumption

For design purposes, the reinforced soil block is considered as a rigid coherent mass. Sandri (1997) observed post-earthquake deformation of GRS retaining walls in the field and noted behaviour, that for vertical walls, could validate the rigid block assumption.

However, both Watanabe et al. (2003) and Sabermahani et al. (2009) observed simple shear deformation within the reinforced soil block for models faced with a FHR panel or wrap-
around facing, respectively. Whilst the predominant mode of failure for the FHR faced models was overturning, Watanabe et al. (2003) note that this actually comprised simple shear along horizontal planes of the reinforced soil.

Further, Watanabe et al. (2003) note that the interaction of the soil and reinforcement requires further clarification. The lack of understanding of the reinforced soil block and simple shear behaviour under seismic excitation impedes the accurate prediction of seismic performance as discussed in Section 2.2.6.

2.5 Summary

Additional factors of seismic design, such as inertial and seismic earth pressures have been discussed. Observed phenomena such as amplification of ground motion, and threshold critical acceleration help to conceptually explain the behaviour of GRS structures, and this has been examined to enable the interpretation of behaviour observed in the current series of model tests.

An accurate measure of structure-specific critical acceleration, combined with valid mechanisms of deformation, and selection of an appropriate acceleration time history can lead to the development of performance-based design methods. However these methods are subject to many limitations as discussed. This thesis investigates particularly the development of deformation within GRS walls under seismic loading.

It was shown that facing type, reinforcement properties and wall inclination strongly impact on the mode of failure, and the mechanisms of deformation observed. However, there has been limited work on the systematic variation of L/H and wall inclination that has had sole focus on its influence on deformation under seismic conditions, and hence on the underlying mechanisms of deformation. Therefore a series of experimental tests is required to further investigate this. Such tests and their interpretation are the topic of this thesis.

References


Howard, R., Kutter, B., and Siddharthan, R. "Seismic deformation of reinforced soil centrifuge models." Geotechnical Earthquake Engineering and Soil Dynamics III, University of Washington, Seattle, Washington, USA.


CHAPTER 3

EXPERIMENTAL DESIGN AND TESTING PROCEDURES

3.1 Introduction

This chapter discusses in detail the experimental model, construction methodology, and testing procedures used in this study. Section 3.2 discusses the model GRS wall design, designed at prototype scale, and geometrically scaled down. Equipment required for design and construction prior to physical model testing is discussed in Section 3.3 and included: a rigid box within which model GRS walls could be built and subsequently tested on the shake-table; sand storage units that were also used for sand deposition into the box; a weighted plate used for compaction of the wall; and a removable bracing system for the GRS wall face. Additionally, a seal to prevent sand leakage around the wall face was also investigated.

Scaling considerations are discussed in the terms of the model geometry, soil, reinforcement, facing and the selection of a model excitation in Section 3.4. The shake-table motion dynamics are shown in Section 3.5, followed by the instrumentation used to measure the dynamic response of the GRS physical model in Section 3.6. The instrumentation included: seven accelerometers, six displacement transducers, and three high-speed cameras used to track soil texture and observe soil deformation through the transparent acrylic sidewall for subsequent analysis using Geotechnical Particle Imaging Velocimetry (GeoPIV) software.
Model construction and experimental set-up are discussed in Section 0. The remaining sections show the static performance of the wall, the as-constructed relative density, and an attempt to measure the fundamental frequency of the models’ soil deposit.

3.2 Wall Design

Full-scale and large-scale testing of GRS walls some 4.8m, 2.8 m and 2.0 m high has been undertaken by Fannin and Hermann (1990), Sakaguchi (1996) and Ling et al. (2005) respectively, however due to material and spacing limitations, the majority of model-scale tests in the literature have been conducted on smaller models of 1 m high or less (El-Emam and Bathurst 2004; Sabermahani et al. 2009; Watanabe et al. 2003). While larger models are able to generate more realistic results than small model-scale tests (Sabermahani et al. 2009), similar limitations on model size, and in particular, those imposed by the dimensions of the University of Canterbury shake-table meant that a model wall height of 900 mm was selected for use in the current tests.

The GRS model wall was designed at prototype scale using the well-established simplified coherent gravity method (FHWA 2001) considering also the experimental objectives. The failure modes considered during design were overtopping and sliding (external), and reinforcement pullout (internal). Reinforcement rupture was not considered. Obviously, the absence of foundation sub-soil and the use of a Full-Height Rigid (FHR) panel facing preclude failure mechanisms such as deep seated slip, bearing capacity, and internal block sliding failures as discussed in Chapter 2. Details of the prototype’s design are given in Appendix A.

A 4.5 m high wall was designed at prototype scale with a typical reinforcement length to wall height ratio, of L/H = 0.75. This is slightly larger than the minimum L/H = 0.70 specified by FHWA (2001). Factors of safety against sliding, overturning and pullout failures were all larger than 2. A geometric scale factor of 1:5 was used to scale the prototype design down to the model height of 0.9 m.

The prototype design required only 5 layers of reinforcement, with a vertical spacing between layers of 750 mm. This spacing is larger than the 500 mm recommended by FHWA (2001) for GRS walls with a wrap-around facing, and equates to a vertical spacing at model scale of 150 mm.
The length of reinforcement is initially defined as 75% of the wall height of 900 mm and is (initially) 675 mm long. The L/H ratio is a key variable throughout testing and will vary from 0.6 to 0.9.

Where possible, appropriate parameters and components of the wall, such as facing, reinforcement, and soil were selected based on the scaling rules suggested by Iai (1989) to satisfy geometric, dynamic and kinematic similitude. This ensured that behaviour at model scale can be used to infer behaviour at prototype scale and is further examined in Section 3.4.

3.3 Experimental Box

A strong rigid box was designed to allow easy construction, testing and dismantling of the reduced-scale model GRS walls. For the purpose of high-speed digital image capture during testing (for subsequent GeoPIV analysis) one side of the box was made of 20 mm thick transparent acrylic. Two further criteria for the box included: a high rigidity and that it would not inhibit the natural behaviour of the sand model (i.e. small boundary effects).

3.3.1 Box design in previous studies

The literature contains many examples of reinforced soil wall physical modelling. These examples were consulted to help determine important parameters for the box design. The predominant consideration concerns that of minimising boundary effects.

Various theories exist as to wall height-to-width ratios (H/W) that minimise frictional boundary effects. Side friction effects can in general be reduced by constructing the model in a box wide enough such that the boundary effects are insignificant at its center-line. Fairless (1989) reported on different wall height-to-width ratios (H/W) for reinforced soil walls that, in theory, minimize boundary effects. He reports Terzaghi (1932) that a wall should be twice as wide as it is high in order to avoid edge effects, and Rowe (1971) who detailed this further stating H/W ratios of 0.5 to 0.3 reduce edge effects. Finally, Fairless (1989) notes that for H/W = 0.5, frictional effects from the boundary will impact on the active earth pressure coefficient, $K_a$, less than 10% and "probably much less."

Box designs reviewed included those by Sakaguchi (1996), Watanabe et al. (2003), and El-Emam and Bathurst (2004). Sakaguchi (1996) built four large-scale model geosynthetic-reinforced soil segmental block faced walls within a box 4.0 m long, 0.9 m wide, and 2.0 m
high (H/W = 2.2) and conducted 1-g shake-table tests that compared the seismic behaviour of three conventional retaining wall structures and one GRS retaining wall. Watanabe et al. (2003) compared three conventional and three FHR panel faced reinforced soil retaining walls scaled by 1:10, again using 1-g shake-table testing. The models were 1.4 m long, 0.5 m high and 0.6 m wide (H/W = 0.83). Both box side walls were transparent allowing deformation to be observed.

El-Emam and Bathurst (2004) conducted multiple 1:6 reduced-scale model shake-table tests that investigated the influence of reinforcement layout and facing geometry on seismic behaviour. The model walls were 1.0 m high and faced by a FHR panel which was isolated from the frictional base and instead was founded on slide rails to enable determination of toe loads and displacements. The wall was built in a box 2.4 m long and 1.0 m wide (H/W = 1.0). The rigid box was lined with plywood and the base covered with glued sand layer that provided friction between the wall and the base.

A summary of the model properties used in each study is provided in Table 3-1.

Table 3-1. Summary of previous model tests’ and this study’s dimensions and materials.

<table>
<thead>
<tr>
<th>Reference</th>
<th>Scale</th>
<th>Box dimensions (L, W, H) (m)</th>
<th>Model height (m)</th>
<th>Height-to-width ratio</th>
<th>Construction materials</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sakaguchi (1996)</td>
<td>1:3</td>
<td>(4.0, 0.9, 2.0)</td>
<td>1.5</td>
<td>1.7</td>
<td>Steel, sand paper glued to base</td>
</tr>
<tr>
<td>Watanabe et al. (2003)</td>
<td>1:10</td>
<td>(2.6, 0.6, 1.4)</td>
<td>0.5</td>
<td>0.8</td>
<td>Steel and plexiglass. Greased Teflon used on side of model wall</td>
</tr>
<tr>
<td>El-Emam and Bathurst (2004)</td>
<td>1:6</td>
<td>(2.4, 1.0, 1.0)</td>
<td>1.0</td>
<td>1.0</td>
<td>Rigid steel lined with plywood and a glued sand base layer</td>
</tr>
<tr>
<td>This study</td>
<td>1:5</td>
<td>(3.0, 0.8, 1.1)</td>
<td>0.9</td>
<td>1.1</td>
<td>Rigid steel lined with plywood and a glued sand base layer, transparent acrylic 20 mm thick</td>
</tr>
</tbody>
</table>

3.3.2 Box design

In addition to an adequate width to reduce boundary effects, the box was designed such that the anticipated failure surface in the model deposit formed during shaking would be contained within the box and not affected by the back wall boundary. This ensured that the effect of the backwall on the model wall was negligible. To predict the theoretical location of a failure surface formed on application of a pseudo-static 0.6g horizontal acceleration, Mononobe-Okabe (MO) theory was used.
The theoretical model was a vertical wall with horizontal backfill, soil with a peak friction angle of 33°, and wall face-soil interface friction angle of 25° (3/4 of the peak friction angle as used by Watanabe et al. 2003). The failure surface was predicted to be 10.7° to the horizontal and was assumed to extend from the wall toe to the backfill surface 5.1 m from the wall. It is unlikely the failure surface would extend this far, considering that the model experiments summarised in Table 3-1 all observed failure surfaces which formed entirely within the strong-box. Hence similar box dimensions are maintained in the current experiments as those noted above and a compromise set the model length to be ~ 2.4 m.

It should be noted that the rigid back wall boundary could reflect seismic energy back into the model. This effect has been reduced in other experiments with the inclusion of a foam barrier to absorb this energy. However, due to the need to calculate an accurate soil deposit density and the construction method chosen, this strategy was not utilized in the present experiment and the model length was assumed long enough to reduce this effect.

A cross section of the box and model inside is shown in Figure 3-1. One side of the box is transparent acrylic (Perspex), allowing high-speed imaging during testing and subsequent Geotechnical Particle Imaging Velocimetry (GeoPIV) analysis.

![Figure 3-1. Experimental setup in Box for L/H = 0.75.](image)

### 3.3.3 Seal design

One important detail of the facing panel is the seal it makes with the box sidewall. The seal should meet two important functions: First it should minimise its influence on model behaviour by generating low friction with the box sidewall; second, it should prevent the
leakage of sand around the wall face. Friction force, \( F_f \), under static loading comprises two components as shown in Equation 3-1.

\[
F_f = \mu F_N
\]  

(3-1)

Where \( \mu \) is the static coefficient of friction, and \( F_N \) is the force normal against the box sidewall. It is therefore apparent that the former can be treated with appropriate selection of materials and the latter through seal design.

Of the research identified above, the seal was mentioned in only the experimental methods reported by Watanabe et al. (2003). Here the seal consisted of a thin strip of foam with Teflon tape wrapped around it and attached to the edge of the facing panel. While Watanabe et al. (2003) did not quantify the friction force, it is assumed negligible. This is because the foam would have generated minimal normal force on the sidewall, and, coupled with the low frictional properties of the Teflon contact, would have resulted in a very low friction force between the wall panel and strong-box sidewall.

Difficulties in fabrication of a wall panel similar to that used by Watanabe et al. (2003) precluded its use and a variety of other designs were trialled with varied success. A variation of that used by Watanabe et al (2003) was selected and consisted of Teflon tape and foam as shown in Figure 3-2.

![Figure 3-2](image.jpg)

Figure 3-2. Plan view of seal against the Perspex sidewall (a) and schematic (not-to-scale) detailing of the seal and friction force components (b). Note that the backup seal is used to prevent leakage (if any) during compaction and is removed for testing.
The seal comprised of two parts. The first part was comprised of a Teflon strip attached to the side of the facing panel and curved to fit the box side wall. A strip of foam was attached on top of this to apply a small normal force on the box sidewall which ensured a good seal. The second part consisted of a ‘backup seal’ and this consisted of a piece of sponge attached to the front of the facing panel. The ‘backup’ seal was forced against the box side wall to prevent any leakage of sand during model construction with a wooden strip. During testing, the wooden strip was removed and the backup seal taped away such that there was no contact between it and the sidewall of the strong-box. The seal designed as above, prevented any serious sand leakage both during model preparation and testing.

Tilt table tests and spring force tests were used to quantify the frictional properties of the seal and friction force of the wall in rotation (see Appendix A). The friction force required to generate rotation was a low $F_r = 1 \text{ N}$, equating to roughly $0.1\%$ of the theoretical total force on the back of the wall (corresponding to the active state). A further comparison of friction force with the earth pressures near the top of the wall was made, as earth pressures are lower near wall top and friction could have a larger impact on wall response. The friction equated to just $1.3\%$ of the earth pressure force acting against the top $200 \text{ mm}$ of the wall. This was again deemed to have a minimal impact on model response.

It should be noted that the seal described above was developed during the process of testing, and was the solution generated during the second half of testing. Tests in which a different seal was used are identified in the results.

### 3.4 Physical model

Important for all reduced-scale model tests is the need to ensure model similitude to allow behaviour at prototype scale to be inferred from that at model scale. In this section, aspects of similitude are defined and used to select (where applicable) model components of soil, reinforcement, reinforcement-soil interface, facing and reinforcement-facing connections. Similitude also applies during testing; hence the selection of model excitation parameters is discussed.

#### 3.4.1 Similitude aspects

In order to model behaviour at prototype scale correctly with reduced-scale models, it is important not only to scale down its geometry, but also to consider stress dependant
behaviour (Sabermahani et al. 2009). Further, the response of the model may be influenced by low confining pressure, ill considered boundary effects and improperly scaled reinforcement mechanical properties (Koseki et al. 2006). Thus Iai (1989) derived similitude specifically for geotechnical applications whereby the basic definitions of effective stress, strain, the constitutive law, overall equilibrium, pore water equilibrium and mass balance at model and proto-type scales are proportional.

The geometric scale factor, $\lambda$, is the proportionality constant between the model (subscript m) and prototype (subscript p) geometry, as shown in Equation 3-2. Similar proportional equations are assumed for other parameters such as stress-strain, overall equilibrium, mass balance and pore water pressure.

\[
(x)_{p} = \lambda(x)_{m}
\]

(3-2)

Inspection of the combined proportionality equations finds that each scale factor can be reduced down to a function of just three independent scale factors, $\lambda$, $\lambda_p$, $\lambda_e$, namely the geometry, saturated soil density, and soil strain scale factors as shown in Table 3-2.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Scaling factor (prototype/model)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density</td>
<td>$\rho$</td>
<td>$\lambda_{\rho}$</td>
</tr>
<tr>
<td>Effective stress of soil</td>
<td>$\sigma'$</td>
<td>$\lambda \lambda_p$</td>
</tr>
<tr>
<td>Length</td>
<td>$x$</td>
<td>$\lambda$</td>
</tr>
<tr>
<td>Strain of soil</td>
<td>$\varepsilon$</td>
<td>$\lambda_{\varepsilon}$</td>
</tr>
<tr>
<td>Displacement of soil and/or structure</td>
<td>$u$</td>
<td>$\lambda \lambda_{\varepsilon}$</td>
</tr>
<tr>
<td>Velocity of soil and/or structure</td>
<td>$u'$</td>
<td>$(\lambda \lambda_{\varepsilon})^{0.5}$</td>
</tr>
<tr>
<td>Acceleration</td>
<td>$u''$</td>
<td>$1$</td>
</tr>
<tr>
<td>Time</td>
<td>$t$</td>
<td>$(\lambda \lambda_{\varepsilon})^{0.5}$</td>
</tr>
</tbody>
</table>

For the present experiments, the geometric scale factor, $\lambda_e$, is equal to 5. However, scale factors relating the saturated soil density, $\lambda_p$, and soil strain, $\lambda_{\varepsilon}$, at model and prototype scales are more difficult to determine. Iai (1989) suggests that for the simple case when $\lambda_p = 1$, i.e. the soil density is the same for both prototype and model scales, then the strain scale factor, $\lambda_{\varepsilon} = \lambda^{0.5}$.

The derived similitude by Iai (1989) was tested in a series of saturated plane strain compression tests and was found to hold for low soil strains and confining pressures 5 -
392 kPa. Thus the similitude is not valid at model failure and/or large shear strains, \( \gamma > 10\% \) at prototype scale. The similitude also has the following assumptions that limit its applicability:

- The soil skeleton is regarded as a continuous medium (reinforcement inclusion makes this invalid).
- The strain is small such that the linear strain approximation \( d\varepsilon = Ldu \) holds and that the equations of equilibrium before and after deformation are the same.

As the main project aim was to investigate the performance of GRS walls prior to failure, data was obtained at low shaking levels and also during and post failure. This initial data is deemed to meet the similitude conditions; whilst the data obtained at large strains corresponding to failure requires special interpretation in light of violating the abovementioned similitude conditions.

Further, the inclusion of reinforcement into the model is obviously necessary, and the effects on the fundamental model similitude conditions are deemed to be minor. However, correct scaling of reinforcement, and reinforcement layout (length and vertical spacing) is important, as these parameters affect the formation and location of failure wedges (Watanabe et al. 2003) and can generate results not indicative of full-scale behaviour (Nova-Roessig and Sitar 2006).

### 3.4.2 Soil

Albany sand was selected for the model soil; it is relatively rounded with the properties specified in Table 3-3. Reasons for its selection included that it is commercially available, that it is a clean sand, and that it generates minimal dust during sand deposition and wall construction.

<table>
<thead>
<tr>
<th>Table 3-3: Albany Sand soil properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>Property</td>
</tr>
<tr>
<td>Solid particle density</td>
</tr>
<tr>
<td>Mean particle size</td>
</tr>
<tr>
<td>Maximum void ratio</td>
</tr>
<tr>
<td>Minimum void ratio</td>
</tr>
<tr>
<td>Peak friction angle(^1)</td>
</tr>
<tr>
<td>Critical state friction angle(^1)</td>
</tr>
</tbody>
</table>

Notes: \(^1\) As determined by Roper (2006)
In geotechnical engineering, the confining stress level is a dominant factor behind soil behaviour (strength, stiffness and strain) and requires correct modelling. For instance, some modes of failure documented in previous 1-g tests have not been observed in the field, due to low model stress levels (Nova-Roessig and Sitar 2006). Thus in order to model accurate prototype soil stress-strain behaviour, the geotechnical centrifuge has been used in some cases (Howard et al. 1998; Ling et al. 2004; Nova-Roessig and Sitar 2006; Viswanadham and Konig 2009), where gravity, and thus soil stress is increased N-times.

However the current tests are to be conducted at 1-g, and special consideration of soil density, vertical confining stress, and stiffness is necessary. Where the same soil material, and thus density, is used for both the prototype and model, Iai (1989) provides the simple case where the density scale factor is 1. Thus effective stress is scaled only by the geometric scale factor of 5. At small strain, the initial (elastic) shear modulus of the soil is dependent on the effective stress of the soil as in Equation 3-3.

\[ G \propto \sigma^{\alpha} \] (3-3)

Where \( G \) is the shear modulus (stiffness), \( \sigma' \) is the effective stress, and \( \alpha \) is some exponent of order 0.5 for sands or 1 for clays. Thus the model soil stiffness at small strains should be reduced from that of the prototype by \((5)^{0.5} = 2.2\) (Wood 2004).

At larger strain associated with permanent deformation, Wood (2004) recommends invoking critical state soil mechanics to ensure representative model behaviour. The schematic in Figure 3-3 shows the initial states of the prototype and model with reference to the critical state line.
Figure 3-3. Schematic detailing critical state soil mechanics and its use in scaling model density (point m) with prototype density (point p) at medium-to-large strains as suggested by Wood (2004)

To achieve similar values of state variable, \( \psi \), the relative change in void ratio, \( e \), from the initial state to the critical state needs to be retained. Assuming a typical sand has a critical state line with a local slope, \( s \), of roughly 0.03 (Wood 2004), and reducing the confining stress between prototype and model scales by the geometric scale factor of 5, then the difference in initial void ratios is calculated as shown in Equation 3-4:

\[
\Delta e = s \ln \lambda
\]  

(3-4)

Thus to ensure similar behaviour between prototype and model scales, the difference in void ratios should be roughly 0.05. Hence for the prototype constructed with Albany sand at relative density, \( D_r = 90\% \), the model should be constructed at \( D_r = 75\% \). Sabermahani et al. (2009) similarly noted that in theory the model should be looser than for the prototype and as such, selected model target relative density's of 47\% and 84\%, lower than that used in the field.

However, any scaling of soil density needs to be balanced with the:

- Desire for good soil-reinforcement interlock;

- Capability of model preparation procedures to generate mid-range (50 – 75\%) densities consistently;

Hence it was deemed simpler to create a model of sufficiently high density with minimal variation in density across the test series. A target relative density of \( D_r = 90\% \) was selected. Section 3.9 shows the density achieved during model construction.
3.4.3 Reinforcement

Modelling of geosynthetics for similitude is difficult; contrary to soil, the same material often cannot be used (Viswanadham and Konig 2004). Viswanadham and Konig (2004) consider two aspects of the reinforcement that require appropriate scaling: 1) frictional bond behaviour of the reinforcement-soil interface, and 2) the tensile strength–strain behaviour of geosynthetics. El-Emam and Bathurst (2004) report that stiffness, not ultimate tensile capacity, governs deformation behaviour, and is thus one of the most important parameters to model correctly. This is in contrast to current design codes that select reinforcement based on the ultimate tensile capacity of the reinforcement.

Geosynthetic-reinforcement available in New Zealand is manufactured by Stratagrid and Tensar, supplied by Stevenson's and Maccaferri respectively. Reinforcement selection was confined to these makes and based on the best value for stiffness similitude. Table 3-4 shows the stiffness of various geogrid products.

<table>
<thead>
<tr>
<th>Manufacturer</th>
<th>Product</th>
<th>Use</th>
<th>Stiffness measure</th>
<th>Value (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensar</td>
<td>UX1800HS</td>
<td>Reinforced Soil</td>
<td>J₅%</td>
<td>1900</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Slopes (RSS)</td>
<td>J₂%</td>
<td>2375</td>
</tr>
<tr>
<td></td>
<td>Uniaxial 3326</td>
<td>Tunnels</td>
<td>Eᵣ(J₀%)</td>
<td>1400</td>
</tr>
<tr>
<td></td>
<td>UX1000HS</td>
<td>RSS</td>
<td>J₅%</td>
<td>460</td>
</tr>
<tr>
<td></td>
<td>Biaxial 3326</td>
<td>Tunnels</td>
<td>E</td>
<td>321</td>
</tr>
<tr>
<td>Stratagrid</td>
<td>Microgrid</td>
<td>RSS</td>
<td>J₂%</td>
<td>220</td>
</tr>
</tbody>
</table>

ᵃᵇ Axial stiffness at 5%, 2% strain respectively
ᶜ Initial elastic axial stiffness (i.e. stiffness at 0% strain)
ᵈ Calculated by assuming stiffness degradation of 25% occurs between 2% and 5%.

Tensar UX1800HS was selected as comparable to typical reinforcement stiffness at prototype scale (El-Emam and Bathurst 2004). UX1800HS has an axial stiffness, J₂% = 2375 kN/m. To determine an appropriate corresponding model scale stiffness the similitude rules proposed by Iai (1989) were used to determine the stiffness scale factor as in Equation 3-5.

\[
J_m = \frac{J_p}{\lambda^2} = \frac{2375}{5^2} = 95 \text{ kN/m} \quad (3-5)
\]

Where \( J_p \) and \( J_m \) are the axial stiffness at prototype and model scale respectively, and \( \lambda \) is the geometric scale factor. Therefore the model scale stiffness (at 2% strain) should be roughly 95 kN/m.

Nakajima et al. (2007) performed scale-model tests on two models of different reinforcement
stiffness, using a polyester and a phosphor bronze reinforcement. The axial stiffness of each reinforcement was determined by direct tension testing, and at 2% axial strain, the axial stiffness was approximately 185 kN/m and 60 kN/m respectively. Even though the material properties were largely different, upon testing, the models demonstrated little observed difference in seismic performance (Nakajima et al. 2007).

Therefore Microgrid was selected for the model reinforcement. Whilst the axial stiffness of Microgrid is 220 kN/m and is approximately double that of the similitude derived stiffness of around 95 kN/m, the difference in stiffness, and order of magnitude, is similar to that of the abovementioned experiment by Nakajima et al. (2007), and was thus deemed to model the reinforcement sufficiently.

Microgrid is a polyester geogrid manufactured by Stratagrid, with the relevant design properties detailed in Table 3-5.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Units</th>
<th>Testing method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial stiffness, ( J_{2%} )</td>
<td>220</td>
<td>kN/m</td>
<td></td>
</tr>
<tr>
<td>Ultimate Tensile Strength, ( T_{ult} )</td>
<td>29.2</td>
<td>kN/m</td>
<td>ASTM D 4595</td>
</tr>
<tr>
<td>Creep Limited Tensile Strength</td>
<td>18.5</td>
<td>kN/m</td>
<td>ASTM D 5262</td>
</tr>
<tr>
<td>Long term design Tensile Strength (LTDS)(^a), ( T_{sl} )</td>
<td>14.0</td>
<td>kN/m</td>
<td></td>
</tr>
</tbody>
</table>

\(^a\) LTDS or \( T_{sl} \) is based on the ultimate tensile strength, \( T_{ult} \) reduced by reduction factors due to creep, installation damage, and durability.

The Microgrid reinforcement stiffness (\( J_{2\%} \)) is higher than that used in previous research by Sabermahani et al. (2009) and El-Emam and Bathurst (2004) of 0.09 - 29 kN/m and 90 kN/m respectively. Hence the Microgrid is considered an extensible reinforcement at prototype scale; its higher stiffness at model scale may mean the reinforcement behaves more like a non-extensible reinforcement.

Reinforcement extensibility is an important reinforcement parameter which alters the formation of lateral earth pressures within the reinforced soil block. FHWA (2001) defines an extensible reinforcement as one where “The deformation of the reinforcement at failure is comparable to or even greater than the deformability of the soil”. This means that loading induces a strain response in the reinforcement which allows the soil to relax and induce the active earth pressure condition. In contrast, an inextensible reinforcement (larger axial stiffness) resists the formation of active earth pressure conditions; rather the earth pressure approximates the at-rest earth pressure conditions, especially within the upper sections of a
GRS wall (FHWA 2001). In light of this, lateral earth pressure conditions at model scale need to be interpreted carefully.

### 3.4.4 Soil-Reinforcement interaction

The soil-reinforcement interface is the second issue for appropriate reinforcement scaling and selection as specified by Viswanadham and Konig (2004). The bond strength between the soil and reinforcement comprises frictional and passive resistance, which are functions of interface friction, reinforcement geometry and vertical confining stress (which has been discussed in Section 3.4.2 above).

Frictional resistance is determined by the interaction between the soil and reinforcement. The soil-reinforcement interaction is defined by manufacturer specified coefficients for pullout and direct sliding, determined via pullout testing in different soils. Microgrid has a soil-reinforcement interaction coefficient of 0.8 – 0.9 for a uniformly-graded Sand or Silty Sand such as Albany sand. Thus this coefficient is the same at both prototype and model scale because the same materials have been used.

To scale the passive resistance, i.e. the interlock of soil particles with the reinforcement, the reinforcement grid mesh size and characteristic soil particle size diameter (PSD) requires scaling. A comparison between the PSD of reinforced soil backfill recommended in the NZ Guidelines (Murashev 2003) and the PSD of Albany sand is shown in Figure 3-4. The NZ Guidelines provide upper and lower bounds on the PSD of soil suggested for use in the reinforced zone, with a mean $D_{50} = 4.35$ mm. Albany sand has a $D_{50} = 0.3$ mm, roughly 15 times smaller, and, in terms of prototype scale, is over-scaled by 3 times ($15/5$).
Figure 3-4. Comparison of reinforced backfill recommended PSD from the NZ Guidelines (Murashev, 2003) and Albany Sand used in the tests.

The Microgrid mesh size is 7.5 by 2.5 mm, with a mesh area of ~ 19 mm². Compared to a typical geogrid mesh area of 3300 mm², the Microgrid mesh is smaller by 175 times. This means that the reinforcement does not meet the requirements of scaled geometry for passive resistance. Viswanadham and Konig (2004) notes that this has been a factor in past experimental testing and that researchers have been more concerned with modes of failure and qualitative rather than quantitative comparisons. However, Nova-Roessig and Sitar (2006) report research conducted by Richardson and Lee (1975) where sufficient soil-reinforcement interlock in the scale model was only achieved through setting the reinforcement at an impractical L/H = 2.5. As noted by Watanabe et al. (2003) above, this would impact on failure patterns and modes observed.

The above analysis indicates that the mean grain size to reinforcement mesh size has not been geometrically scaled well, and this could impact on observed behaviour due to a reduced passive resistance. However it was deemed as being of secondary importance to reinforcement stiffness, reinforcement-soil interlock, and reinforcement L/H, to obtain results more consistent with those observed at prototype scale.
3.4.5 Full-height Rigid (FHR) facing

The facing is more difficult to scale quantitatively, as there is large variety in how facing structures are designed, constructed and used. In these tests, Full-Height Rigid (FHR) facings were used due to their ‘excellent’ performance in previous earthquakes in Japan (Tatsuoka 2008).

El-Emam and Bathurst (2005) considered the effects of facing condition by constructing the wall face from rectangular steel cross sections bolted rigid together. The sections could be rotated and stacked to double wall thickness from 36 to 72 mm, with a doubling of facing weight also. Facing geometry and type had a large influence on wall behaviour; very stiff facings transferred up to 60% of dynamic reinforcement loads to the facing toe. The results indicated decreasing stability with increasing wall mass for the ranges tested; thus a stiff panel of low weight was selected for the current experiments.

The facing comprises an aluminium panel nominally 960 mm by 800 mm and 5 mm thick, stiffened in the vertical and horizontal directions with steel angles. The entire face weighs 30 kg and was constructed with the aim that its influence on behaviour was minimal, and that facing rigidity was maintained during preparation and testing. This ensured that the conditions of a typical FHR facing panel were achieved, without other characteristics such as the weight and geometry influencing results considerably.

3.4.6 Connections

The mechanical properties of the connection govern the transfer of stress from the reinforcement to the wall face and visa versa. Connections between the reinforcement can either be rigid, in which case the full benefits of a FHR panel facing are realised, or imperfectly rigid. For the tests conducted, a rigid connection was created with a steel strip clamping the reinforcement to the face. This prevented slippage of the reinforcement, simplifying interpretation of results.

3.4.7 Model excitation

The three important components of an earthquake motion for engineering purposes are the acceleration amplitude, frequency content and its duration (Kramer 1996). With reference to
the similitude discussed previously, motion frequency was scaled appropriately as discussed below. The effects of amplitude and duration are considered in the model results.

A typical medium to high intensity strong motion earthquake has a predominant frequency of 2 – 3 Hz (0.3 – 0.5 seconds) (Hatami and Bathurst 2000). As noted previously, Iai (1989) allowed for the simple case where the model soil density is the same both at prototype and model scales, and the soil strain scale factor $\lambda_e$ is unity. The similitude rules govern the selection of an appropriate frequency of input motion at model scale as shown in Equation 3-6:

$$(t)_p = \lambda_t (t)_m = (\lambda \lambda_e)^{0.5} (t)_m$$  \hspace{1cm} (3-6)

Where $(t)_p$ and $(t)_m$ are the time dimensions at prototype and model scale, $\lambda_t$ is the time scaling factor, $\lambda$ the geometric scale factor, and $\lambda_e$ the soil strain scale factor. Equation 3-7 calculates the predominant frequency at model scale:

$$(t)_m = \frac{(t)_p}{(\lambda \lambda_e)^{0.5}} = \frac{0.5}{(5)^{0.5}} = 0.2\text{sec}$$ \hspace{1cm} (3-7)

Hence a predominant frequency of 5 Hz was selected, allowing comparison of results with other model studies conducted at this frequency of excitation (El-Emam and Bathurst 2004; El-Emam and Bathurst 2005; Watanabe et al. 2003).

For modeling purposes, the shape of the driving motion is also important. A series of studies conducted by Watanabe et al. (2003) showed that the intensity of loading increased in the order of: shaking with an irregular time-history, shaking sinusoidally and pseudo-static loading with a tilt-table. Whilst an irregular time-history is more realistic, its analysis and repeatability to enable comparisons across tests is difficult, and a sinusoidal motion was chosen for its high repeatability and easier interpretation of results. As discussed later, this simple base excitation contains more energy than an earthquake time history of similar predominant frequency and peak acceleration amplitude (El-Emam and Bathurst 2005; Watanabe et al. 2003). In other words, in a sinusoidal excitation, the PGA occurs every cycle, while an irregular time history scaled to the same PGA, might only have a single occurrence of this PGA. Hence, in terms of duration of time at PGA, the sinusoid is a more intense excitation than the irregular ground motion.

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For completeness, a sinusoidal excitation of 5 Hz frequency with an acceleration amplitude of 0.1g for a duration of 10 seconds (50 cycles) was applied for the first shaking step. The acceleration amplitude was then increased in 0.1g increments until model failure.

3.4.8 Summary of model geometry

The previous sections detail the similitude considerations behind the selection of various model parameters and geometry. These issues are summarised in Figure 3-5 (a) and (b).

![Diagram](image)

Figure 3-5. Model geometry inside strong box for: a) vertical model reinforced the longest at L/H = 0.9, and b) inclined wall at 70°, reinforced at L/H = 0.75.

The wall face is a FHR aluminium panel 5 mm thick and 960 mm high which fits into the 800 mm wide box mounted onto a 4.0 m long by 2.0 m wide shake-table. (Note the extra 60 mm wall facing in height is to allow the same panel to be used for the inclined model wall). The backfill is Albany S sand compacted to a target relative density of $D_r = 90\%$ deposited upon the base of the rigid box with a glued sand layer to ensure sufficient friction between the rigid base and backfill. The sand deposit consists of six layers up to a total height of 900 mm.
Based on stiffness similitude Microgrid reinforcement was selected. This reinforcement is a polyester geogrid and can be classified as ‘extensible’ at prototype scale. The reinforcement length was varied to achieve a reinforcement-to-wall height ratio, L/H = 0.6, 0.75 and 0.9. The vertical spacing between reinforcement layers for all models was 150 mm. Figure 3-5 (a) shows the longest reinforcement length of 810 mm (L/H = 0.9).

Figure 3-5 (a) and (b) also shows the reinforcement numbering used for the description of model deformation in Chapter 5.

3.5 Shake-table and motion dynamics

The box and GRS model within were mounted on the University of Canterbury shake-table. The uni-directional shake-table is dimensioned 4.0 m by 2.0 m, has a maximum velocity of 240 mm/s, maximum payload of 20 T and a peak displacement amplitude stroke of ±120 mm.

The shake-table is displacement controlled; displacement amplitude and frequency with time generate the desired acceleration as shown in the governing dynamic Equation 3-8.

\[
a(t) = -\omega^2 d(t) \\
\omega = 2\pi f
\]

(3-8)

Where \( a(t) \) and \( d(t) \) are the shake-table acceleration and displacement, and \( \omega \) and \( f \) are the angular and temporal frequencies of motion. Thus a sinusoidal wave of frequency 5 Hz and displacement amplitude of 1 mm creates a sinusoidal acceleration with an amplitude of 0.1g.

The experimental sign convention is positive for outward displacements, velocities and accelerations (with respect to the backfill) and is shown in Figure 3-6 below. The minus sign in Equation 3-8 means that the acceleration and displacement vectors are always 180° out of phase, that is, outward wall accelerations are positive and occur when the wall is negatively displaced.
The shake-table system is driven by a 280 kN double-acting hydraulic actuator, powered by a 300 Hp motor operating at 4000 psi. The hydraulic actuator is controlled by a set of two E072-054 servovalves controlled by a TestStar control system supplied by MTS Systems Corporation. The system has a built in Linear Variable Differential Transducer (LVDT) that measures table displacement. Ang (1985) reviews the shake-table capabilities in-depth.

The shake table at the University of Canterbury has an unloaded resonance frequency of around 17.5 Hz (Murahidy 2004). Murahidy (2004) further developed a relationship between shake-table payload and natural frequency. This relationship was used to predict the natural frequency of the shake-table, box and sand mass of the current testing system. The box filled with sand had a combined weight of 3600 kg, and according to Murahidy’s (2004) relationship, the combined system had a natural frequency of around 13.7 Hz; larger than the testing frequency of 5 Hz. Little can be done to mitigate possible resonance effects, however they are considered minimal.

3.6 Instrumentation

Instrumentation was designed to effectively quantify the seismic response at varying intensity levels and is shown in Figure 3-7 (a - c).
Figure 3-7. Model instrumentation for (a) the vertical wall reinforced the longest at L/H = 0.9 (Test-3 and 5); b) plan view of vertical wall (reinforced by L/H = 0.9); and, c) the inclined wall, reinforced by L/H = 0.75 (Test-7).

3.6.1 Facing displacements

Two arrays of three displacement transducers (Disp. 1 – 6) were located in series at the front of the wall at heights 775, 500 and 200 mm as seen in Figure 3-7 (a) and (c). Each array was located 200 mm from the side-wall and named the North (Disp. 1 – 3) and South (Disp. 4 – 6) Array, as seen in Figure 3-7 (b).

3.6.2 Shake-table and soil accelerations

Four accelerometers to be placed into the soil deposit at varying heights were purchased for the experiment. A number of factors influenced their selection which included:
- **Size** (minimal impact on natural vibration of the soil)
- **Ability to withstand the abrasive sand environment as well as high soil pressures**
- **Measurement range and safe overload capacity**
- **Natural frequency**
- **Data sampling rate.**

Accelerometer model AS-2GB manufactured by Kyowa was selected as it met all of the above listed criteria. The compact accelerometer weighed 25 g and is dimensioned nominally at 14 x 14 x 20 mm resulting in minimal impact on the vibration mode of the soil deposit.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rated Capacity</td>
<td>± 9.807 m/s²</td>
</tr>
<tr>
<td>Frequency Response (at 23°C)</td>
<td>DC to 60 Hz (± 5%)</td>
</tr>
<tr>
<td>Resonance Frequency</td>
<td>100 Hz</td>
</tr>
</tbody>
</table>

The accelerometers were mounted on aluminum plates measuring nominally 50 mm by 80 mm and 3 mm thick, with the total height of the accelerometer being 14 mm off the plate’s surface. The cable is integrated with the accelerometer perpendicular to the plate surface and extended roughly 15 mm above the accelerometer.

![Kyowa AS-2GA accelerometer mounted on aluminium plate.](image)

Figure 3-8. Kyowa AS-2GA accelerometer mounted on aluminium plate.

Figure 3-7 (a) and (b) shows the location of the accelerometers within the soil deposit and on the shake-table and top of the box. In total, six accelerometers were used in the experiment. Acc. 1 and Acc. 6 were mounted on the shake-table and the rigid box itself, respectively. Acc. 2 - 4 were located vertically in line 250 mm from the wall face, and along the box centre-line to reduce boundary effects, and were used to measure the reinforced soil block.
response. For the inclined wall, the accelerometers were still placed vertically and on average, 250 mm from the wall face. Acc. 5, was used to quantify the far-field response and was positioned along the box centerline, 800 from the box back wall (again to minimise boundary effects).

3.6.3 High-speed camera imaging

Three high-speed cameras were used to record deformations during testing. Camera 1 focused on the reinforcement layer R4 within the reinforced soil block and Camera 2 focused on the interface between the reinforced and retained backfill zones. Camera 3 was used to capture the global deformation of the entire wall. Figure 3-9 shows the regions recorded by each camera. Cameras 1 and 2 are visible in the picture mounted on tripods.

![Image](image_url)

Figure 3-9. Regions recorded by each camera are shown in as the white dashed lines for Camera 1 – C1, Camera 2 – C2, and Camera 3 – C3 (not shown).

Camera 1 is a SVSi Camera (USA), and Camera 2 and 3 are MotionPro X3 Cameras (USA). The camera properties are summarised in Table 3-7. High-speed Nikkon lenses were used for Cameras 1 and 2, to ensure sufficient light for texture tracking, whereas a standard Nikkon lens was able to be used for Camera 3, which operated at a slower frame rate.
<table>
<thead>
<tr>
<th>Camera</th>
<th>Purpose</th>
<th>Resolution (pixels)</th>
<th>Frame Rate (fps)</th>
<th>Exposure time (msec)</th>
<th>Acquisition time (s)</th>
<th>Colour</th>
</tr>
</thead>
<tbody>
<tr>
<td>Camera 1</td>
<td>Reinforced soil block</td>
<td>512 x 1280</td>
<td>200</td>
<td>5</td>
<td>12</td>
<td>Monochrome</td>
</tr>
<tr>
<td>Camera 2</td>
<td>Interface between reinforced and retained backfill</td>
<td>1280 x 1024</td>
<td>200</td>
<td>5</td>
<td>12</td>
<td>Monochrome</td>
</tr>
<tr>
<td>Camera 3</td>
<td>Global deformation</td>
<td>1280 x 1024</td>
<td>100</td>
<td>10</td>
<td>12</td>
<td>Colour</td>
</tr>
</tbody>
</table>

The nature of high-speed photography means that the images recorded are stored within the camera RAM during image capture. Hence the limited RAM of each camera controlled the selection of frame rate, resolution and acquisition duration. At the completion of each shaking step, the acquired image data was transferred to hard disk; this process could take up to 15 minutes between shaking steps.

### 3.7 Model construction

The staged construction procedure for GRS walls with FHR panel was discussed in Section 1.4.2. The model wall was not constructed in the staged manner for full-scale walls as described by Tatsuoka (2008). Rather, the construction method braced a single aluminium panel against the box, and the sand was layered and compacted behind it. A similar method was used by El-Emam and Bathurst (2004) who argued it to be a construction technique somewhat similar to the field case of an incrementally constructed (unbraced) segmental (modular block) wall and a FHR panel process.

Because soil behaviour depends on parameters such as confining stress, packing and density, the most important aspect of model construction is soil placement. Two methods considered for the placement and compaction of soil were those reported by Watanabe et al. (2003) and El-Emam and Bathurst (2004). Watanabe et al. (2003) constructed 500 mm high models in layers by air-pluviation to achieve a relative density of $D_r = 90\%$. El-Emam and Bathurst (2004) constructed their 1000 mm high models by first distributing sand in 100 mm thick layers, and then vibrating the shake-table and box after each layer to achieve a relative density of $D_r = 86\%$. Because air-pluviation is time intensive and requires the use of robotic-automated equipment the latter method used by El-Emam and Bathurst (2004) was selected for the current experiments and is described in Section 3.7.3 below.

Therefore, the staged construction of the model wall involved a number of steps: Connection of reinforcement to the FHR aluminium panel face; bracing the wall face for construction; layered placement and compaction of backfill; reinforcement placement; accelerometer
placement; incorporation of marker sand in both vertical and horizontal lines; and finally, the un-bracing of the wall face to produce the static self-weight condition for subsequent testing. Throughout the construction, layer-by-layer measurements of soil height and mass were made to enable calculation of average soil density. The movement of the wall during un-bracing was also recorded. These stages are described in the following sections.

3.7.1 Reinforcement connections

It is important to ensure the connection between reinforcement and facing panel is rigid (Section 3.4.6 above). This ensures that the wall deformations measured are the result of soil and reinforcement interaction as opposed to some (un-measured) degree of reinforcement slippage at the facing connection, thus simplifying the analysis.

The reinforcement is first connected to the FHR aluminium panel prior to being braced in the box. Five steel strips 16 mm wide and 3 mm thick were used to provide the rigid mechanical connection. The reinforcement was cut with an extra 16 mm in length which was clamped in between the facing and steel strip and bolted at 100 mm centres, as shown in Figure 3-10. Nakajima et al. (2007) used a similar method for reinforcement connection.

Figure 3-10. Rigid mechanical connection between reinforcement and aluminium panel.

The strength of the reinforcement at the connection sometimes limits design strength achievable in the reinforcement, and FHWA (2001) recommends long-term testing of this strength should this be a factor. No physical testing on the connection was conducted; however on visual inspection of the connection after deconstruction, slippage was never identified and the rigid assumption appeared validated.
3.7.2 FHR panel bracing

The FHR facing was an aluminium panel dimensioned 960 mm by 798 mm and 5 mm thick. The panel was stiffened symmetrically in the vertical direction by 4 steel angles with cross section dimensions of 41 mm by 41 mm and 15 mm thick. Three further steel angles of the same dimensions were used to stiffen the face in the horizontal direction at heights of 120, 100 and 730 mm. The panel and stiffeners are shown in Figure 3-11.

![Figure 3-11. FHR aluminium panel with attached stiffeners and guides for wall bracing: a) side view, and b) front view.](image)

The facing was placed squarely within the box and braced by six steel tubes arranged in two rows and three columns. The steel tubes were screwed tightly into guides mounted on the vertical and horizontal angle stiffeners. The wall face is shown fully braced in the box in Figure 3-12 (a). Figure 3-12 (b) shows a schematic plan view of the facing braced within the box.
3.7.3 Construction of model deposit

The soil deposit was constructed in layers 75 mm thick. Prior to sand placement, plastic tubes were taped against the acrylic wall for vertical columns of coloured sand to be poured in, as described in Section 3.7.5 below. The first stage of layer construction involved the determination of the mass of sand necessary to achieve the target density. Slight changes in face position would change the box volume, and thus mass required to achieve target soil density; hence internal length and widths were measured manually once the wall face was braced.

Four storage containers were constructed each able to contain 900 kg of sand. A crane scale calibrated to 1000 kg and accurate to +/- 0.2 kg was connected to the container and crane. The container was lifted above the box and the combined mass of the container and sand recorded (Figure 3-13 (a)). The container sliding trapdoor was opened until the required sand mass for the layer (usually 245 kg) was deposited in the box. The sand was deposited evenly across the wall (Figure 3-13 (b)).
Figure 3-13. Storage container lifted into position above the box to record initial combined mass of container and sand (a) and sand deposition into the box (b).

Figure 3-14 (a – l) shows the construction process up the placement of the bottom layer of reinforcement. Figure 3-14 (a) shows the empty box with vertical tubes in position ready for the first layer of sand to be deposited as described above.

To ensure consistent compaction of each layer of sand, the surface of the sand layer had to be horizontal. Hence the deposited sand (Figure 3-14 (b)) was first raked, and then flattened with an adjustable wooden board, which was dragged across the surface of the layer. Depending on how loose the deposited sand was, the board’s depth (with respect to the top of the box) was adjusted. Normally, the loose sand deposit was flattened to a level approximately 10 mm above the target layer thickness of 75 mm. The sand deposit after this process is shown in Figure 3-14 (c).

a) Empty box with vertical tubes taped into position.
b) Loose sand after deposition.

c) Loose sand after raking, and using the wood panel to make the sand surface flat.

d) The compactor plate is lowered onto Layer 1 of flat loose sand. The shake-table is vibrated at 13 Hz for 10 seconds to compact the sand Layer. Layer height is then measured to the top of the compactor plate at both sidewalls at intervals of 30 cm. The known thickness of the compactor plate is removed to determine the sand layer thickness.

e) Layer 1 post compaction and removal of the compactor plate.

f) Sand is deposited for Layer 2 on top of compacted Layer 1.
g) Layer 2 is then raked and flattened with suspended board to known height prior to compaction.

h) Flattened Layer 2, ready for compaction.

i) Compactor plate on Layer 2, post compaction. Soil height is measured post compaction.

j) Layer 2 post compaction with compactor plate removed.

k) Boards are carefully placed on top of Layer 2 to enable access to the inside of the box. The tape holding the vertical coloured sand lines is carefully removed.
Finally, Layer 2 is scraped flat at the sidewall to correspond with the surface at the box centreline and the reinforcement is pulled out horizontally from the wall face. A coloured sand line is created against the sidewall window (flattened to a thickness of 4 mm), and the vertical tubes reset for the next two layers.

Figure 3-14 (a) – (l). Describes the layer construction process in detail.

Three methods were trialled to compact flat the sand layer:

- Manual hand-held plate compaction;
- Shake-table vibration with no surcharge weights;
- Shake-table vibration with surcharge.

The first two were deemed unsatisfactory: for the first method, it was impossible to uniformly compact layers; and for the second method, the shake-table/box/sand system dynamics were such that, in contrast to the construction methodology used by El-Emam and Bathurst (2004), simple shake-table vibration was found to be unable to compact the sand. The third method, whereby a surcharge weighing approximately 950 kg was used during vibration was selected, and Figure 3-14 (e) shows the compactor plate resting on the flat sand layer. For this method, the shake-table was vibrated with a displacement stroke amplitude of 2 mm at 13 Hz for a duration of 10 seconds. This equates roughly to an acceleration amplitude of 1.4g.

Post-compaction, the height of the compaction plate with respect to target layer heights marked along both sidewalls of the box was measured and recorded at 30 cm intervals. As the compactor plate is rigid, these measurements gave an indication of the average layer compaction response along the centreline of the box. The compactor plate was removed and the process repeated for the second layer of sand as shown in Figure 3-14 (e) – (i). Figure 3-14 (j) – (k) shows the horizontal layers and vertical columns of black sand that were then created as discussed in Section 3.7.5 below.
3.7.4 Reinforcement and accelerometer placement

The reinforcement was placed before the horizontal coloured sand layer was made and the reinforcement pulled taut ready for deposition of the next sand layer. Care was taken not to impart any tension to the reinforcement, as this could alter the stress condition of the model prior to shaking as shown in Figure 3-15 (a).

![Figure 3-15. Reinforcement unfolded from the wall and placed in position on the surface of layer 2 (a), accelerometer embedded into the top of layer 3 at a distance of 250 mm from the wall face and along the box centreline (400 mm from the box sidewall) (b).](image)

After the compaction of layers 3, 8 and 11, accelerometers (Acc 2, Acc 3, and Acc 4) were embedded upside down at the top of the layer along the box centreline 250 mm from the wall face as shown in Figure 3-15 (b). Acc 5 was placed 800 from the box back-wall.

3.7.5 Vertical columns and horizontal layers of black marker sand

Use of horizontal coloured sand lines to clarify deformation in some form or another is common in the literature for tilt-table, 1-g shake-table and geotechnical centrifuge tests (see for example Koseki et al. (1998), Watanabe et al. (2003), El-Emam and Bathurst (2004), Nova-Roessig and Sitar (2006)). In particular centrifuge tests by Nova-Roessig and Sitar (2006) used white sand along each reinforcement layer to identify the reinforced zone, green sand layered within the backfill to show the location of potential failure surfaces and black sand markers arranged in a grid to monitor lateral and horizontal displacements. While the grid arrangement used by Nova-Roessig and Sitar (2006) was able to fully resolve visualisation of deformation, using only horizontal coloured marker layers limits visualisation to vertical and rotational movements.
Relatively few researchers have used vertical columns of coloured sand to monitor translational movement deformation within the reinforced-soil block. Lo Grasso et al. (2004) and Howard et al. (1998) utilised coloured columns of black sand in shake-table tests 0.35 m high and centrifuge models 0.33 m high respectively to better visualise mechanisms of failure. This allowed both failure planes within the backfill and horizontal displacement of the reinforced soil zone to be clearly visible. However, these were constructed in models of relatively small dimensions and the use of a similar construction technique for models 900 mm high was found difficult to replicate.

A method for constructing vertical columns of black sand into models almost 3 times taller than that constructed in the abovementioned studies was created for the purposes of this research. Plastic tubes ~ 200 mm in length (longer than the reinforcement spacing of 150 mm) were taped to the inside of the transparent wall as visible in Figure 3-14 (a) – (l) which described the layer construction process in detail. The initial tube set up for the entire layer is shown in Figure 3-14 (a). The tape was looped back up to the top of the tube such that when pulled, the tape would be gradually removed from the bottom of the tube until the tube was no longer taped to the inside of the wall, as shown in Figure 3-16 (a – d).

![Figure 3-16. The tubes are filled with black sand when taped (a), after two layers have been compacted the tape is removed by pulling upwards (b) leaving just the plastic tube held against the wall by the sand pressure (c) which is then removed (d).](image)

Once taped, the tubes could be filled with black sand, a close up of which is shown in Figure 3-16 (a). After two layers have been compacted the tape was removed by pulling upwards (Figure 3-16 (b)) and this left the plastic tube held against the wall by sand pressure only.
Figure 3-16 (c) shows the tube then being manually removed. This staged construction ensured:

- The plastic tubes could be removed (high soil density prevented un-taping of the tubes and subsequent tube removal when tube sections were longer);

- The plastic columns did not interfere with the reinforcement placement at the box sidewalls;

- The soil disturbed on removal of the plastic tubes is able to be subsequently re-densified during compaction of above layers.

During compaction some sand would be vibrated upwards around the compaction plate at the box sidewalls (see Figure 3-14 (h)), this, and the removal of the vertical tubes left the sand in an uneven and loose state at the box sidewalls (see Figure 3-16 (d)); as mentioned above, this sand was scraped flat and the layer surface made flat for subsequent reinforcement placement and horizontal line construction.

The horizontal black sand lines were made against the acrylic wall above the placed reinforcement and across the retained backfill. Each line was made by funnelling the coloured sand; it too was scraped level to achieve a layer thickness of 4 mm. Once the reinforcement was placed, the vertical tubes were re-positioned above ready for construction of the next two layers as shown in Figure 3-14 (l). While care was taken to ensure that the columns were placed vertically in-line to enable inter-layer displacement to be visible, sometimes during compaction the sand layers beneath that being compacted underwent some lateral displacement. This resulted in the vertical lines sometimes being slightly curved prior to testing, as can be seen in photos in subsequent sections. Thus only qualitative measures of deformation can be inferred from the black marker lines.

3.8 End of construction and static self-weight response

Upon completion of compaction of all layers, the facing braces were removed. At this point, some movement of the wall occurred (on the order of 0.5 – 1.5 mm at wall top) as lateral earth pressures reduced towards the active earth pressure, $K_a$, and the reinforcement was fully/partially engaged. The initial displacement is governed primarily by the stiffness of the reinforcement (inextensible vs extensible) and connections, the amount of slack in the
reinforcement, and the compaction of the retained fill. The displacement of the wall upon the removal of the braces was recorded at each displacement transducer and is shown in Figure 3-17 for each reinforcement layout: L/H = 0.6, 0.75 and 0.9 (Tests-6, 1 and 5 respectively). Note that for Figure 3-17 (a) and (c), the points of lateral displacement are averaged from the North and South Arrays, and also that Figure 3-17 (c) shows the results from both Tests-3 and 5. This position of the wall under self-weight was taken as the initial position prior to shaking.

Note that the lines linking lateral displacements are extrapolated to both the wall toe and top. From these extrapolations - which may not be valid at such small displacements - both sliding and overturning modes of deformation are apparent.

The largest displacement upon removal of the bracing occurred at the top of the wall. It can be seen that a decrease in facing displacement from 1.5 mm to 1.1 mm at wall top occurred with an increase in L/H from 0.75 to 0.9. No difference in facing deformation is noticeable for reinforcement ratios of L/H = 0.6 and 0.75. Factors that may act to reduce the movement of the wall under static self-weight loading include:

- Minimal slackness of reinforcement present during construction
- The retained fill density is high
- The reinforcement is considered extensible at prototype scale, however as noted is likely under-scaled at model scale in the current experiments (i.e. the model reinforcement acts inextensibly).
Permanent facing displacement upon removal of braces (mm)

Figure 3-17. Average lateral displacement of the wall face under self-weight at the end of construction of walls a) Test-6, L/H = 0.6; b) Test-1, L/H = 0.75; and c) Tests-3 and 5, L/H = 0.9.

The deformations equate to displacements at the wall top of around ~ 0.17% of the wall height for L/H = 0.6 and 0.75 (Test-6 and 1), and around ~ 0.11% for L/H = 0.9 (Tests-3 and 5). Ling et al. (2004) measured facing displacements during and post-construction of a full-scale 6 m high GRS segmental-panel faced retaining wall. The largest lateral facing displacements under static self-weight were recorded at mid-height (due to the non-rigid segmental face) and were of the order of 0.5% of the height, larger than those observed in this study (Figure 3-17).

The NZ Guidelines (Murashev 2003) provide an empirical assessment of lateral displacement post construction based on the FHWA (2001) Guidelines. Equation 3-9 is used to calculate a maximum lateral displacement of the wall assumed to occur at the top of the wall during construction.

\[ \delta_{\text{max}} = \delta_r H / 75 \]  

(3-9)
Where \( \delta_r \) is the relative lateral displacement (dimension-less) based on reinforcement layout for a 6 m high wall, and H (m) is the height of the wall under consideration. An empirical design chart determines the relative lateral displacement, \( \delta_r \), for reinforcement ratios \( L/H = 0.6, 0.75 \) and \( 0.9 \) as 1.22, 0.73, and 0.95 respectively. Thus the method predicts a maximum lateral displacement, \( \delta_{\text{max}} \) during construction to be approximately 9 mm, 11 mm and 15 mm for the 0.9 m high model walls.

The discrepancy between that predicted and observed is large and is probably due to the empirical derivation being based on 6 m high walls. This, and the difference with observations made by Ling et al. (2004) are due to the in-exact nature of modelling of reinforcement and soil properties as detailed above.

### 3.9 Model deposit density

Noted in Section 3.4.2 is the importance and influence of soil density on stiffness and thus model response. Hence the measurement of the model deposit density is necessary to ensure consistency of soil parameters across tests.

The compaction process uses two plates that apply a uniform vertical stress of \(~ 5.3 \text{ kPa}\) across the soil surface under static conditions. The box, braced wall, soil, and plates are then vibrated by the shake-table at 13 Hz with acceleration amplitude of 1.4g for 10 seconds. Due to densification of the soil underneath, the plate typically drops \(~ 8 \text{ mm}\) from its pre-compaction position and its final height with respect to the bottom of the box is recorded. In this way, the soil layer’s height and thus volume, combined with its known mass is used to determine an average relative density for the layer using Equations 3-10 and 3-11.

\[
\rho_d = \frac{m_s}{V_T}, \quad \rho_s = \frac{\rho_s}{1 + e}, \quad \text{therefore, } e = \rho_s \frac{V_T}{m_s} - 1
\]  \tag{3-10}

\[
D_r = \frac{e_{\text{max}} - e}{e_{\text{max}} - e_{\text{min}}} \times 100(\%) \tag{3-11}
\]

Where \( m_s \) is the mass of the deposited soil for one layer, \( \rho_d \) is the soil layer dry density, \( \rho_s \) is the solid particle density, \( V_T \) is the total volume of densified sand, \( e \) is void ratio, \( e_{\text{max}} \) and \( e_{\text{min}} \) are the maximum and minimum void ratios for Albany sand (defined in Table 3-3 above), and \( D_r \) is the relative density of the soil. All walls had a target relative density,
$D_r = 90\%$. The New Zealand Geotechnical Society (2005) thus defines the model as being ‘very dense’. The average density for the entire deposit for each test is shown in Table 3-8.

<table>
<thead>
<tr>
<th>Test</th>
<th>$m_t$ (total kg)</th>
<th>$\rho_d$ (kg/m$^3$)</th>
<th>e</th>
<th>$D_r$ (%)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test-1</td>
<td>2982.0</td>
<td>1738</td>
<td>0.525</td>
<td>94</td>
<td></td>
</tr>
<tr>
<td>Test-2</td>
<td>2956.5</td>
<td>1708</td>
<td>0.551</td>
<td>93</td>
<td></td>
</tr>
<tr>
<td>Test-3</td>
<td>2939.4</td>
<td>1705</td>
<td>0.554</td>
<td>92</td>
<td></td>
</tr>
<tr>
<td>Test-4</td>
<td>2939.4</td>
<td>1700</td>
<td>0.559</td>
<td>90</td>
<td></td>
</tr>
<tr>
<td>Test-5</td>
<td>2946.2</td>
<td>1706</td>
<td>0.554</td>
<td>92</td>
<td></td>
</tr>
<tr>
<td>Test-6</td>
<td>2938.2</td>
<td>1697</td>
<td>0.562</td>
<td>89</td>
<td></td>
</tr>
<tr>
<td>Test-7</td>
<td>2842.0</td>
<td>1715</td>
<td>0.545</td>
<td>95</td>
<td>Inclined wall</td>
</tr>
</tbody>
</table>

Notes: $^1$ Maximum error for the entire deposit is determined as below and equals $\sim 3.3\%$.

Table 3-8 shows that the total mass used for each test are similar in value and this suggests consistent preparation procedures and repeatability of density. While similar to the density achieved by Watanabe et al. (2003), it is higher than for tests conducted by other researchers. El-Emam and Bathurst (2004) constructed walls of $D_r = 86\%$, and Sabermahani et al. (2009) of $D_r = 47$ and 84%. Further, the relative density is higher than some large-scale tests conducted by Ling et al. (2005). In these tests, the walls were 2.8 m high and constructed to $D_r = 52 - 56\%$.

### 3.9.1 Sensitivity study

Calculation of relative density is sensitive to volume and mass measurements, particularly when considering a single lift of 75 mm. For instance, consider a single layer of thickness 75 mm in height with width and length of 800 mm and 2410 mm respectively, and with mass of 245.2 kg. A measurement error of +/-1 mm height (1.3% inaccuracy), results in a 7.1% change in the relative density, from $D_r = 90\%$ to 83.6%.

Sensitivity to mass measurement is similar. The typical mass used in each layer is 245 kg with a known accuracy of +/-0.2 kg. Again, for a single layer, the possible error results in a relative change of 0.6% from $D_r = 90.4\%$ to 89.9%. The errors considered are only two, acting in isolation, of those listed in Table 3-9.
Table 3.9. Measurement quantities and possible errors

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Quantity</th>
<th>Possible error</th>
<th>$\Delta D_r (%)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mass / layer (kg)</td>
<td>245</td>
<td>+/- 0.2</td>
<td>-0.5</td>
</tr>
<tr>
<td>Height / layer (mm)</td>
<td>75</td>
<td>+/- 2</td>
<td>-13.9</td>
</tr>
<tr>
<td>Length / layer (mm)</td>
<td>2408</td>
<td>+/- 2</td>
<td>-0.4</td>
</tr>
<tr>
<td>Width / layer (mm)</td>
<td>798</td>
<td>+/- 2</td>
<td>-1.3</td>
</tr>
<tr>
<td>Total Mass (kg)</td>
<td>2940</td>
<td>+/- 2.4</td>
<td>-0.1</td>
</tr>
<tr>
<td>Total deposit Height (mm)</td>
<td>900</td>
<td>+/- 2</td>
<td>-1.3</td>
</tr>
<tr>
<td>Solid density (kg/m$^3$)</td>
<td>2650</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>$e_{\text{max}}$</td>
<td>0.83</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>$e_{\text{min}}$</td>
<td>0.53</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Maximum error in $D_r$

<table>
<thead>
<tr>
<th></th>
<th>per layer</th>
<th>entire deposit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Delta D_r$</td>
<td>-16.1</td>
<td>-3.7</td>
</tr>
</tbody>
</table>

The above parametric study considers a single layer. As construction progresses the calculations of an average relative density for the entire wall become progressively less sensitive, and the final average density for the entire wall is determined as shown in Table 3-8. (There is less relative error with increasing wall height and mass). A maximum worst-case error (when errors are all additive) for the entire wall is calculated as an absolute error in terms of relative density measurement of 3.7%.

Density impacts on the shear modulus which determines acceleration amplification which in turn, has important effects on the magnitude of the lateral earth pressure coefficient (Steedman and Zeng 1990). Thus the uniformity of density with depth is also of interest, and a density calculation for each sand layer is necessary.

However, this is made difficult because, during construction, compaction of layers higher up (and increasing confining pressure with increasing wall height) most likely act to further compact layers below, similar to the concept of under-compaction proposed by Ladd (1978). It should be noted that further densification of lower layers may not always be the case, and that some loosening could occur (as shown in Figure 3-18 below).

In order to gain a measure of this change, the change in elevation of the horizontal black lines (spaced vertically at 150 mm intervals) were measured after compaction of each layer, in order to measure the change in thickness of soil layers below. Figure 3-18 shows the relative density of each layer during and at completion of the wall. Height readings were made at the front of the wall (dark points) and at 1400 mm from the wall face (grey points). Obviously, measurements were confined to the acrylic window and density changes at the back of the wall could not be so measured.
Figure 3-18. Relative density of each layer during and after construction. Post compaction of each layer, layer heights were recorded at two locations and are plotted separately: dark for density measurements at the face, and grey for density measurements at mid-length (1400 mm) along the wall. Note each measurement has a possible absolute 11% error.
Figure 3-18 shows the relative density of each layer measured at the front of the wall, and midway along the wall, with further compaction of layers above. Considering the change in relative density of the bottom most layer, the trend shows increasing density with further compaction steps. In general, the same trend can be seen for other layers, except for L3 - L4 and L9 - L10 where there is an apparent decrease in relative density during compaction of above layers above.

After construction was completed, the relative densities of each layer are spread across the range $D_r = 74$ to $98\%$. No trend was observed relating a layer's relative density with its elevation (L7 - L8 have the highest relative density, whilst L9 - L10 have the lowest). This would indicate that the density of the soil deposit is non-uniform. However, on average, the relative density for the entire Test-6 wall is $D_r = 89\%$ as specified above.

The inconsistent trend observed for the change in relative density during construction, and the range in final density values achieved, highlights the poor accuracy in the measurement technique. This is because the layer thickness used in the calculation was based upon the elevation of the horizontal coloured sand lines, and these were observed to deform slightly during each vibration compaction. It shows that the density of individual layers is difficult to quantify precisely and that the technique used can not be relied upon.

However, given that one of the most important parameter governing GRS walls behaviour is the stiffness, $G$, within the wall, it is unlikely that the recorded non-uniformity in relative density will impact largely on wall response.

### 3.10 Model fundamental frequency

The fundamental frequency of the model is important to quantify, as this identifies possible resonance conditions that could lead to overamplified response and premature failure (El-Emam and Bathurst, 2004). Additionally, Hatami and Bathurst (2000) state that retaining walls of typical height ($H < 10\, \text{m}$) are considered as short-period structures and therefore, their seismic response is dominated by their fundamental frequency.

The fundamental frequency of a soil profile is given by Equation 3-12 and is dependant on the shear wave velocity and soil profile thickness.
\[ f_f = \frac{v_s}{4H} \]  

(3-12)

Where \( f_f \) is the fundamental frequency; \( v_s \) is the shear wave velocity and \( H \) is the soil deposit thickness (in this case equal to 0.9 m). The shear wave velocity is defined by the relationship given in Equation 3-13 and the initial shear modulus, \( G \), at small strain levels, is in turn proportional to the effective stress level of the wall as shown in Equation 3-14.

\[ v_s = \frac{\sqrt{G}}{\sqrt{\rho}} \]  

(3-13)

\[ G \propto \sigma^\alpha \]  

(3-14)

Where \( \rho \) is the soil density, \( \sigma \) the effective stress level, and the exponent \( \alpha \) is a constant of about 0.5 for sands (Wood 2004).

However, descriptive parameters such as the fundamental frequency, shear wave velocity and model density that describe the state of the soil deposit, change once the soil deposit is subjected to an earthquake motion. This process involves first strain-induced degradation of soil stiffness, \( G \), and a resultant change in shear wave velocity which in turn affects the fundamental frequency of the model.

Nova-Roessig and Sitar (2006) investigated this effect during a series of centrifuge tests on model-scale GRS walls that were subjected to different scaled earthquakes of varying intensity. In order to quantify earthquake-induced densification of backfill, photographs were used to make an estimate of volume change during testing and from this a density change. From initial relative densities of 55% and 75%, changes in relative density of up 44% were recorded. Further, low-amplitude step displacements were applied intermittently between earthquake motions which enabled changes in the shear wave velocity caused by the earthquake-induced densification to be quantified. Up to a 36% increase in shear wave velocity was recorded indicating that the abovementioned effects are significant.

In model tests of ~ 1.0 m height and relative densities ~ 46 to 86% surveyed in the literature the fundamental frequency was typically 22 Hz (+/- 0.5 Hz) (see Sabermahani et al., 2009; El-Emam and Bathurst, 2004 for examples).
Other parameters, such as reinforcement spacing, length and reinforcement stiffness have been found to have minimal influence on model natural frequencies (Nova-Roessig and Sitar 2006). This is verified by numerical modelling which showed facing condition to also have minimal influence (Hatami and Bathurst 2000).

3.10.1 Impulse test

To determine the fundamental frequency of the model, generally a small acceleration pulse is applied to the base of the soil deposit (to induce shear waves) and the natural vibration frequency of the model is recorded via accelerometers placed within the backfill. This was the methodology used by El-Emam and Bathurst (2004), Nova-Roessig and Sitar (2006) and Sabermahani et al. (2009). The pulse should be small enough such that it is within the elastic range of the soil stress-strain curve so that no permanent deformation occurs; as discussed above, a degraded condition would give an erroneous fundamental frequency for the original model.

It was deemed difficult to apply an acceleration pulse small enough to prevent permanent deformation from occurring using the shake-table. Hence, for the current tests, another method was trailed. This method employed an accelerometer-instrumented hammer which was used to strike the box in order to generate a small acceleration pulse within the soil deposit. The results of this method are summarised herein.

It was impossible to generate an acceleration pulse from below, and the hammer was instead used to strike the back-wall of the box at around mid height of the soil deposit. The back-wall was 2.4 m from the wall face, 2.15 m from the vertical accelerometer array in the reinforced zone (Acc 2, 3, and 4), and the impact was parallel to the direction of accelerometer measurement. Two impacts of roughly 3.5 kN and 4.4 kN impulses spaced roughly one second apart were made to ensure a full frequency content; the impact force-time history is shown in Figure 3-19.
Figure 3-19. Hammer impact time history.

Because the hammer struck the box back-wall, an acceleration pulse in the longitudinal direction was induced within the soil deposit. This impulse would be transmitted by a longitudinal wave with velocity, $v_p$. It is likely that shear waves within the soil deposit would also be induced, and these are transmitted by velocity, $v_s$. The ratio of the two different velocities is shown in Equation 3-15:

$$\frac{v_p}{v_s} = \sqrt{\frac{2 - 2\nu}{1 - 2\nu}}$$

(3-15)

Where $\nu$ is poisson's ratio, where a typical ratio for sand is 0.3. Hence the ratio between the longitudinal wave and shear wave velocity is calculated as approximately 1.87.
Figure 3-20. Acceleration response during hammer test. Acc 5 records the first pulse as it is closest to the back-wall. However there is also a difference in arrival time between accelerometers located in vertical array (Acc 2, Acc 3, Acc 4).

The time histories of each accelerometer are shown in Figure 3-20. After the initial hammer pulse which peaked at 0.8535 seconds, Acc 5, located 800 mm from the back-wall recorded an acceleration impulse at 0.8595 seconds. This was followed by various delays until pulse arrival was recorded by the accelerometers located in vertical array near the front of the wall. Acc 2 recorded the pulse at 0.8610 seconds.

Care must be taken to determine whether the accelerometers recorded longitudinal wave or shear wave velocity, as this affects the interpretation of arrival times. Because longitudinal waves travel faster, it was assumed that the first pulse recorded by the accelerometers was the result of an induced longitudinal wave.

The delay time between the initial hammer pulse, and its recorded arrival, coupled with the known longitudinal distance between the accelerometers and the back-wall can be used to estimate the longitudinal wave velocity, $v_p$. In turn, the shear wave velocity and hence the fundamental frequency of the wall can be estimated via Equations 3-15 and 3-14, respectively. Various estimates for the fundamental frequency are shown in Table 3-10.
Table 3-10. Estimation of fundamental frequency from the longitudinal shear wave velocities calculated via the delay time between the first hammer impulse and that recorded within the backfill.

<table>
<thead>
<tr>
<th>Instrument</th>
<th>Impulse arrival (s)</th>
<th>Delay (s)</th>
<th>Longitudinal distance (m)</th>
<th>Calculated ( V_s ) (m/s)</th>
<th>Calculated ( V_p ) (m/s) *</th>
<th>Calculated ( f_r ) (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hammer</td>
<td>0.8535</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Acc 2</td>
<td>0.3610</td>
<td>0.0075</td>
<td>2.16</td>
<td>106.7</td>
<td>199.5</td>
<td>44.9</td>
</tr>
<tr>
<td>Acc 3</td>
<td>0.3620</td>
<td>0.0085</td>
<td>2.16</td>
<td>254.1</td>
<td>475.2</td>
<td>107</td>
</tr>
<tr>
<td>Acc 4</td>
<td>0.3635</td>
<td>0.01</td>
<td>2.16</td>
<td>216.0</td>
<td>403.9</td>
<td>90.9</td>
</tr>
<tr>
<td>Acc 5</td>
<td>0.3595</td>
<td>0.006</td>
<td>0.8</td>
<td>133.3</td>
<td>249.3</td>
<td>56.1</td>
</tr>
</tbody>
</table>

Notes: * \( V_p \) is calculated assuming poisson’s ratio, \( \nu = 0.3 \).

The calculated fundamental frequency ranges from 44.9 Hz – 107 Hz. As Acc 2 recorded the pulse first, it stands to reason that this data is the most reliable record for longitudinal wave velocity. This is because data from accelerometers higher up could be complicated by interference from shear waves originating from lower layers (Acc 2 was the lowest accelerometer at an elevation of 215 mm). Hence a fundamental frequency of approximately 44.9 Hz was assumed.

To check this result, the free-vibration acceleration response of the four accelerometers embedded within the soil deposit was recorded, and the fourier spectrum plotted in Figure 3-21. The figure plots the free-vibration frequency response for the 5 second period recorded, and not just the arrival of the first pulse. It could be assumed that the response includes natural vibration due to both longitudinal and shear wave excitation.
Figure 3-21. Free-vibration Fourier spectrum of the embedded accelerometers

Figure 3-21 shows that a peak in the Fourier amplitude response occurred for all accelerometers at 41.0 Hz. This value agrees reasonably well with the 44.9 Hz determined previously based on the arrival time of the impulse longitudinal wave. Because Figure 3-21 shows this value for all accelerometers, and does not include the approximations made to derive Table 3-10, it could be assumed as a more accurate value for the fundamental frequency of the soil deposit.

This value is almost double that of the fundamental frequency determined in the experimental studies by El-Emam and Bathurst (2004) and Sabermahani et al. (2009). However, it should be noted that both these studies had lower model densities ($D_r = 46\%$ to $86\%$) and larger model heights of $1 \text{ m}$, and that both these parameter values contribute to a reduced fundamental frequency in comparison with that determined for the current tests. It should be noted that these effects are unlikely to contribute to such a large discrepancy.
Hence, the above results are somewhat questionable and show the inadequacy in attempting to measure the fundamental frequency via an impact at the back of the wall. However, the results do confirm that the fundamental frequency of the wall is likely to be significantly higher than the excitation frequency of 5 Hz used during testing. As noted, this reduces the likelihood of an overamplified model response and premature failure.

In the future, a pure shear wave introduced via a small acceleration pulse at the base of the wall (via the shake-table) should be used to more accurately quantify the fundamental frequency of the soil deposit.

### 3.11 Summary

To conduct shake-table tests on reduced-scale model GRS walls, a strong box was designed and constructed after consulting the literature of typical reduced-scale experimental testing of retaining walls.

Utmost attention was given to various details of the model and preparation procedures. In particular, a methodology to compact the soil deposit and achieve consistent relative density was presented in detail. This involved the soil deposit being constructed in layers and vibration compacted with the shake-table. A weighted plate was placed on top of the soil deposit to aid in the compaction. Additionally, the FHR facing panel and the bracing method used during construction of the wall were described.

Horizontal coloured ‘marker’ lines of sand were incorporated into the soil deposit and a technique to incorporate vertical ‘marker’ lines of sand was developed. These enabled deformation patterns to be visible within the soil deposit.

The similitude rules as developed by Iai (1989) and Wood (2004) were considered in order to appropriately scale the soil, reinforcement and GRS wall design to model testing conditions. The reinforcement selected was a Stratagrid Microgrid, supplied by Stevensons Limited. The Microgrid reinforcement is slightly under-scaled in terms of its stiffness and over-scaled in terms of mesh area to reinforcement area.

The model excitation for testing on the University of Canterbury shake-table was based on similitude considerations for the frequency, and a desire observe behaviour at increasing levels of base acceleration. The excitation consisted of a sinusoidal motion at frequency 5 Hz, duration 10 seconds, with steadily increasing amplitude in 0.1g increments. This
frequency is significantly lower than that of the natural frequency of the model deposit, which was found to be ~ 41 Hz (using the free-vibration method).

Seven reduced-scale model GRS walls were constructed and tested with different L/H ratios and wall inclination. A high relative density was achieved for all walls which ranged from 89% to 95% with a possible maximum error in the average relative density for deposit of 3.3%. It was found that there was a certain degree of non-uniformity within the models; however given the high relative density achieved it is unlikely that this will have a major impact on wall response.

The experimental instrumentation consisted of: 6 accelerometers, with 4 of these placed within the soil deposit to record the dynamic soil response within the reinforced soil block and the 'far-field'; 6 LVDT’s to record wall face displacement; and 3 high-speed cameras to obtain deformation within two selected areas and globally.

The results of the test series are presented and discussed in Chapter 4.

References


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CHAPTER 4

EXPERIMENTAL RESULTS

4.1 Introduction

A series of seven reduced-scale geosynthetic-reinforced soil wall model tests were conducted using the University of Canterbury shake-table. These tests investigated the influence of reinforcement L/H ratio and facing inclination on the seismic performance of GRS walls. Three of these tests had the same reinforcement layout as other tests, in part to ensure consistency of the construction methodology and repeatability of the testing apparatus, and also due to some errors in the testing protocol.

The first part of this chapter, Sections 4.3 and 4.4, concentrate on the typical results of one of these tests, Test-6. Test-6 has been selected because the construction methodology and testing procedure had been perfected allowing the most representative results of GRS behaviour under seismic loading to be obtained. Section 4.3 illustrates the shake-table input motion and presents a selection of the resulting facing displacement and acceleration time histories during Test-6. These are followed by a discussion of the deformation and failure modes observed. Section 4.4 considers typical analysis of the Test-6 raw data providing further information of deformation pre-failure and at failure. Acceleration amplification is also calculated and compared using two different methods.

The second part of this chapter utilises results from all tests to analyse the effect of the reinforcement ratio L/H and facing inclination on wall deformation. Section 4.5 presents raw data from the entire test series. Section 4.6 then examines this data using the analysis
methods developed in Section 4.4 to investigate the influence of the reinforcement ratio L/H, and wall inclination on wall response. Displacement-acceleration curves, observed modes of failure and acceleration amplification are compared. One measure of stability, critical acceleration, is observed using four different criteria and compared. A comment on repeatability is also made.

Finally, Section 4.7 examines some of the implications of the test findings to design practice.

4.2 Testing summary

A series of seven reduced-scale model tests on GRS walls with a FHR facing was conducted on the shake-table. For each test, the reinforcement L/H ratio and the inclination of the wall was varied. Three tests were repeated to ensure construction consistency and also due to issues surrounding the wall seal with the box sidewall, and testing error. Table 4-1 shows how these two parameters were varied throughout the testing programme.

<table>
<thead>
<tr>
<th>Test</th>
<th>L/H ratio</th>
<th>Wall angle (°)</th>
<th>Acceleration at failure (g)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test-1</td>
<td>0.75</td>
<td>90</td>
<td>0.6</td>
<td>Seal leakage and brittle failure of cellotape</td>
</tr>
<tr>
<td>Test-2</td>
<td>0.6</td>
<td>90</td>
<td>0.5</td>
<td>Seal leakage (Testing initiated at 0.3g*)</td>
</tr>
<tr>
<td>Test-3</td>
<td>0.9</td>
<td>90</td>
<td>0.7</td>
<td>Testing initiated at 0.6g, 10 Hz</td>
</tr>
<tr>
<td>Test-4</td>
<td>0.75</td>
<td>90</td>
<td>0.65</td>
<td>Seal issue (high friction). Results not used.</td>
</tr>
<tr>
<td>Test-5</td>
<td>0.9</td>
<td>90</td>
<td>0.7</td>
<td>-</td>
</tr>
<tr>
<td>Test-6</td>
<td>0.6</td>
<td>90</td>
<td>0.5</td>
<td>-</td>
</tr>
<tr>
<td>Test-7</td>
<td>0.75</td>
<td>70</td>
<td>0.7</td>
<td>-</td>
</tr>
</tbody>
</table>

NB: * Testing was initiated at a larger acceleration than planned to reduce the quantity of sand lost around the sides of the facing panel once testing was initiated.

Table 4-1 shows that all tests failed within the range of 0.5g and 0.7g base input acceleration. Two of the repetitions (Tests-6 and 2, and Tests-5 and 3) showed that the wall failed at the same base input acceleration, indicating consistency in the construction methodology. Test-4, an intended repeat of Test-1, failed 0.05g higher at 0.65g due to its high friction seal. This issue is discussed in Section 4.5.1, and resulted in all of the results from Test-4 being considered unreliable and hence they were not used for analysis.

A detailed summary of all tests, including testing issues, is made in Section 4.5.1.
4.3 Results of Test-6 – A typical case

4.3.1 Test-6 model wall

Of the seven reduced-scale tests conducted, Test-6 was selected to show in detail representative results of GRS model behaviour under seismic loading. This is because the construction methodology and testing procedure had been perfected, compared to earlier tests.

Test-6 was a vertical wall with reinforcement ratio, L/H = 0.6. This reinforcement layout was the lowest L/H ratio tested. A comparison with Test-5 (reinforced at L/H = 0.9) provides an upper and lower bound on the model seismic response for the range of parameters being tested. The models reinforced at a median L/H = 0.75 (i.e. between that used for Test-5 and Test-6) did not generate results within these upper and lower bounds due to issues surrounding the facing seal, as discussed in Section 4.5.1.

As for all tests, the Test-6 model was 900 mm high, and constructed in the strong box 800 mm wide. The model was 2410 mm long from the wall face to the back wall of the strong box. The model dimensions and the instrumentation used is shown in Section 3.6.

The Test-6 model deposit was constructed to an average relative density of 89%, with the deposit incorporating vertical and horizontal black coloured sand lines to enable better visualisation of failure mechanisms. The as-constructed model prior to testing is shown in Figure 4-1, which also identifies the reinforced soil zone and retained backfill.
The base input shaking, facing deformation and accelerometer time histories recorded during Test-6 are presented in the following sections.

4.3.2 Base input shaking

A sinusoid input motion with predominant frequency of 5 Hz was selected to simplify interpretation of results and enable qualitative comparison with other research. As noted in Section 3.4.7, the selected motion contains more energy than a typical earthquake record of similar predominant frequency and amplitude. This is because the time duration of the peak acceleration is larger in a sinusoidal motion than an irregular acceleration time history (Watanabe et al. 2003).

Hatami and Bathurst (2000) determined that the model response is critically dependant on the structure’s fundamental frequency. The selection of 5 Hz as the frequency of input motion was based on two considerations: firstly that the frequency of shaking be sufficiently lower than the model structure fundamental frequency determined in Section 3.10 as ~ 41 Hz to minimise a possible ‘resonance’ condition; and secondly, model similitude as presented in Section 3.4.7.

The motion duration of 10 seconds corresponds to a long duration earthquake, but was selected so that deformation at different acceleration amplitudes could be observed. Finally, the peak acceleration amplitude was steadily increased in a series of stepped acceleration
sinusoids of 0.1g increments, enabling model response data at various levels of acceleration input to be obtained. The staged testing for Test-6 is shown in Figure 4-2.

![Input base acceleration (g) vs Time (s) for staged testing](image)

**Figure 4-2. Summary of staged testing procedure.**

The test was terminated when the wall reached the limit of the displacement transducers, which had approximately 200 mm of the travel at the front of wall.

The base acceleration of the model was controlled by the displacement time history of the shake-table. The equations of motion, discussed in Section 3.5, describe the corresponding input acceleration. A displacement transducer and accelerometer mounted on the shake-table recorded the dynamic motion of the shake-table during testing and acted as a check on the acceleration specified.

The raw shake-table displacement time history for the first load step of 0.1g for Test-6, and the recorded acceleration time history are shown in Figure 4-3 (a). Figure 4-3 (b) shows the fast fourier spectrum transformation of the raw data, and Figure 4-3 (c) shows the filtered time histories. An in-depth discussion of the filter and corrections applied to raw time history data is provided in Section 4.3.4.
Figure 4.3. The first shaking step of Test-6 at 0.1g: (a) Raw shake-table acceleration and displacement time history data, (b) Fast Fourier Transformation, and (c) filtered shake-table acceleration and displacement time history data.
Figure 4-3 (a) shows the precision of the shake-table system achieved at the lowest acceleration amplitude of 0.1g which is equivalent to a displacement amplitude of +/- 1 mm. As discussed previously, the shake-table acceleration amplitude is controlled through displacement amplitude. It can be seen that the shake-table displacement is asymmetric, and the peak-to-peak, or double amplitude, is approximately 2.1 mm.

The raw acceleration data is shown to overshoot the +/- 0.1g target, to around 0.21g. Figure 4-3 (b) shows the raw data to contain 15 and 25 Hz frequency components (explained in Section 4.3.4), which are of minimal consequence for engineering purposes. Hence these frequencies are filtered using an 8th order, 10Hz low-pass Butterworth filter. Figure 4-3 (c) shows the filtered acceleration record which overshoots the +/- 0.1g target by only approximately 0.01g in the positive and negative directions.

The difference between the target acceleration amplitude and the actual measured acceleration of the shake-table is approximately 10% of the acceleration amplitude. This discrepancy is deemed acceptable as the testing methodology ensures that an exact measure of the actual input acceleration time history is made for evaluation of the wall response.

The Fast Fourier Transformation of the input base acceleration is also shown in Figure 4-3 and indicates that the motion consisted predominantly of 5 and 15 Hz components, with further smaller amplitudes at 25 and 35 Hz. The motion contains little other noise and enables an easier study of the results generated, than, say, an irregular time history with rich frequency content.

The Test-6 model underwent successive increasing base accelerations up to 0.5g at which point the wall failed, providing deformation information over five shaking steps. Displacement of the wall face, accelerations within the soil deposit, and high-speed camera images were recorded during each acceleration step. Additionally, at the end of each shaking step, photographs showing the global deformation and settlements of the wall crest were taken. A selection of typical data obtained is presented in the following sections.

### 4.3.3 Step-by-step deformation

Modes of deformation were limited to overturning and sliding (external deformation), and pullout (internal deformation) of the wall.
Figure 4-4 (a – e) shows the formation of these failure modes with the image sequence taken at completion of 0.1g, 0.2g, 0.3g, 0.4g and 0.5g shaking steps, in (a) through (e) respectively.

The predominant mode of deformation of Test-6 was by overturning. This involved the reinforced soil block rotating about the toe, coupled with multiple external failure surfaces which formed within the backfill behind the reinforced soil block. Also evident is some sliding of the base of the wall along the hard foundation, which occurred largely in the final shaking step of 0.5g during wall failure (Figure 4-4 (e)).
Figure 4-4. Deformation of Test-6 visible at the end of acceleration inputs of 0.1g, 0.2g, 0.3g, 0.4g, and 0.5g upon which the wall failed. The numbered dashed lines highlight the location and progression of failure planes once they become readily discernible by the naked eye.
As seen in Figure 4-4 (a) there is limited deformation evident on completion of 0.1g shaking. On completion of 0.2g shaking (b), an external planar failure surface becomes visible within the top third of the model backfill behind the reinforced soil block (see Section 3.4.8 for reinforcement layer numbering); this is shown by the dashed line number 1. Increased shaking of 0.3g (c) generates a further failure surface that forms deeper within the backfill (dashed line number 2). This failure surface initiated from a point approximately mid-depth along the previously formed (higher) failure plane 1, and extends to the back of the reinforced soil block at around the location of reinforcement R2 tip. This is perhaps evidence of progressive failure and is discussed later in Section 5.3.4. During 0.4g shaking (d) the failure surface initiated at 0.3g is further developed with wedge sliding along the discontinuity and overturning of the wall face.

Finally, upon application of 0.5g shaking (e), a third and lower external failure surface (dashed line number 3) is initiated at the soil deposit surface, and propagates down through the backfill until it reaches the bottom layer of reinforcement, R1. This failure surface then propagates horizontally along the soil-reinforcement interface towards the wall face, and meets another failure surface formed between the wall face toe and the bottom layer of reinforcement, R1. The failure surface which formed at the toe would, in a conventional gravity-type retaining wall, continue up through the backfill until it reaches the wall surface; the inclusion of horizontal reinforcement effectively stopped the reinforced soil block from being compromised in this manner. Similar behaviour was commented on by Watanabe et al. (2003).

During the 0.5g shaking, the reinforced soil block rotated about the toe generating maximum lateral displacement at the top of the wall face. This was coupled with some sliding at the bottom of the wall face along the hard foundation. The failure wedge formed by the lowest external failure surface within the backfill then moved down and horizontally into the gap left by the reinforced soil block. As a result, the highest settlement recorded was 108 mm, and occurred some 300 mm behind the reinforced soil block.

Also visible in Figure 4-4 (e) is an inclined failure surface that traces the back of the reinforced soil block and extends from the soil surface down to the lowest layer of reinforcement, R1.

The deformation described during Test-6 was typical for all reinforcement layout and facing inclination tested in this series. However, differences in reinforcement layout and facing
geometry led to differences in the progression of deformation pre-failure. These differences in behaviour are discussed in the parametric studies of Section 4.5.

4.3.4 Acceleration-time histories

Six accelerometers were used in total, and their location is shown in Figure 3-7. Three Kycwa AS 2GB accelerometers were located within the reinforced zone at the top of soil layers L3, L8 and L11, 250 mm from the wall face; they are named Acc. 2, Acc. 3 and Acc. 4 respectively. Acc. 2 to 4 record the soil response at different elevations within the reinforced soil block. A fourth accelerometer, Acc. 5, is located 1610 mm from the wall face (800 mm from the box back-wall) and records the ‘far-field’ backfill response. All accelerometers are placed along the centre line of the box to minimise side-wall boundary friction effects. Acc. 6 is fastened securely at the top of the box to check whether the box behaves rigidly.

The accelerometers have a measurement precision of +/- 0.002g, significantly lower than the smallest PGA applied to the model, with a possible error of 2% for the lowest PGA of 0.1g used. This suggests a high reliability of the accelerometer data for the current application.

An example of raw acceleration data recorded by Acc. 4 within the reinforced soil block during the final shaking step of 0.5g for Test-6 is shown in Figure 4-5 (a). The response contains noise due to the accelerometer instrument (minor) and various other sources. For engineering applications, the consequence of these high-frequency spikes is minimal as the duration of which they occur is so small. Thus an 8th order low-pass Butterworth filter is used to filter frequency components higher than 10 Hz from the acceleration record. The raw and filtered time histories are shown in Figure 4-5 (a and b). The filtering process results in a slight phase shift to the right and reduction in peak acceleration values recorded.
Figure 4-5. Acceleration time history recorded by Acc 4 for Test-6 during 0.5g shaking step and the data processing applied: (a) raw accelerometer data, (b) filtered accelerometer data using an 8th order 10 Hz low-pass Butterworth filter, and (c) filtered data after baseline correction.

During testing, the accelerometers record the surrounding soil's response which comprises predominantly rotation, as well as some sliding components. Rotation of the accelerometer compromises its ability to measure accelerations in the horizontal (x) plane as some component of acceleration due to gravity is included. Figure 4-6 shows schematically the effect accelerometer rotation has on the measurement of acceleration in the horizontal plane.
Figure 4-6. Correction of accelerometer tilt

Where $a_0$ is the acceleration recorded before shaking, $a_i$ is the acceleration recorded during shaking, $a_x$ is the horizontal acceleration component during shaking, and $\theta$ is the tilt angle of the accelerometer with the horizontal plane. If the angle of the accelerometer tilt with the horizontal is small, then it can be assumed that $a_i = a_x$. However, in some cases the angle was not small and the accelerometer recorded accelerations of up to 0.05g prior to shaking. This corresponded to a tilt of the accelerometer of up to $2.9^\circ$ to the horizontal.

Thus in addition to the filter applied in Figure 4-5 (b), a correction for the initial accelerometer tilt is made using Equation 4-1 for all accelerations records.

$$a_x = (a_i - a_{i0}) \times \cos^{-1}(a_{i0})$$  \hspace{1cm} (4-1)

During shaking, the accelerometer can undergo further rotation, and this contributes to the acceleration not returning to zero after the shaking has stopped. This residual acceleration is baseline corrected with a linearly increasing increment over the duration of shaking to return the residual acceleration to zero.

Selected acceleration-time histories of Test-6 for shaking steps 0.1g, 0.3g and 0.5g are presented in Figure 4-7 to 4.9. The complete set of corrected and filtered acceleration time histories, and filtered facing displacement time-histories, for all tests, are presented in Appendix B.
Figure 4.7. Acceleration time histories for 0.1g shaking step.
Figure 4-8. Acceleration time histories for 0.3g shaking step.
Figure 4-9. Acceleration time histories for 0.5g (final) shaking step.
The acceleration records from Acc. 1 and Acc. 6 were observed to be the same in all tests and confirm that: 1) the box is connected rigidly to the shake-table, and 2) that it does not modify the input motion (due to non-rigid behaviour).

Except for acceleration time histories recorded during 0.1g base input acceleration, accelerometers placed in vertical series within the reinforced soil zone show an increase in the input base acceleration with increasing wall elevation. This could be attributed to the wall face "rocking" about the wall toe, increasing the acceleration amplitude with increasing elevation. In contrast, no increase in acceleration was recorded by Acc. 5 located near the top of the backfill soil; likely due to its increased distance from the deformation at the wall face.

4.3.5 Facing displacement-time histories

During each shaking step, the wall face oscillates and steadily moves in the outwards (positive) direction. This displacement was recorded by two vertical arrays of displacement transducers located 400 mm apart and 200 mm inwards of their respective North and South side-walls. Both arrays consisted of three displacement transducers numbered Disp 1, 2 and 3 and Disp 4, 5 and 6 in series located at heights of nominally 770 mm, 500 mm and 200 mm from the wall base as shown in Section 3.6.1. Assuming the wall remained rigid during the test, these three transducers enabled displacements at the wall top and bottom to be calculated.

Figure 4-10 to Figure 4-14 show the displacement time histories for all shaking steps 0.1g, 0.2g, 0.3g, 0.4g and 0.5g. The total wall displacement at any time is a combination of cyclic, or recoverable displacement, and residual displacement as the wall deforms permanently outwards (El-Enam and Bathurst, 2004). The cyclic component is clearly dependant on the amplitude of shaking, while the residual component is a function of both the amplitude and duration of shaking. The residual displacement at the completion of 0.1g to 0.5g shaking steps increased from 2 mm to 150+ mm respectively.
Figure 4-10. Facing and shake-table displacement time histories at 0.1g shaking step.

Figure 4-11. Facing and shake-table displacement time histories at 0.2g shaking step.
Figure 4-12. Facing and shake-table displacement time histories at 0.3g shaking step.

Figure 4-13. Facing and shake-table displacement time histories at 0.4g shaking step.
The first cycle of the 0.1g shaking step caused a large residual deformation immediately upon application, which was followed by a reduced response for the rest of the displacement-time history. This behaviour was evident throughout all model walls during the first shaking step and is most likely due to some displacement necessary to completely engage all reinforcement layers. That is, during construction there is some small slack in the reinforcement placement, which, in order for all reinforcement to act in unison and resist deformation of the wall face, must first be removed. The displacement-time history demonstrated non-linear behaviour with decreasing incremental residual displacement throughout the rest of the shaking step, such that the residual displacement seemed to almost plateau at a displacement of 1.75 mm.

The first shaking step also showed evidence of slight asymmetry of movement from the longitudinal direction; there is a 0.35 mm discrepancy in residual wall movement between Displacement transducers 1 and 4 (Figure 4-10). No further movement out of plane was generated during increasing base input acceleration.
In the 0.2g and 0.3g shaking step shown in Figure 4-11 and Figure 4-12, wall deformation was irregular though increased steadily throughout the shaking. An increased response can be seen to occur every third cycle generating an increased recovery of displacement back in the negative direction, and an increased displacement in the positive outward direction of almost double that of the trend seen in the other two cycles.

While the shake-table displacement time history is steady, the base input acceleration of the 0.3g shaking step (Figure 4-8) reveals some small irregularities which appear to have been amplified in the facing displacement response. Amplification of the acceleration response of the wall face has previously been recorded by El-Emam and Bathurst (2007) and it is reasonable to presume that this may translate into an amplification of displacement as well.

The irregular displacement response clearly visible in Figure 4-12 was also exhibited at other shaking levels (it appears “washed-out” for the 0.5g shaking step as shown above) and all other tests. A possible explanation is discussed further in Section 4.3.6.

The 0.4g shaking step (Figure 4-14) shows the residual displacement to be less than that which occurred in the previous lower amplitude shaking at 0.3g. This was also evident in other tests and could be the result of two possible effects: the mode of deformation, and the geometry of wall displacements (i.e. overturning vs sliding). These possible effects are discussed below with reference to Figure 4-15.

It can be seen in Figure 4-4 above that the model undergoes predominantly overturning and some sliding. This failure geometry is evident in the facing displacement time histories with the largest displacement occurring at the top most displacement transducers (Disp. 1 and 4). To remove the influence of failure geometry on the facing displacement time histories, the smaller sliding component of deformation (with some overturning component still present) can be somewhat isolated if we consider the displacement time history of the lowest displacement transducer, Disp. 3 or Disp. 6. Figure 4-15 shows the entire facing displacement time history of Disp. 3 (the lowest North Array displacement transducer located at an elevation of 200 mm) created by connecting the Disp. 3 time histories of Figure 4-10 to Figure 4-14.
Figure 4-15. Facing displacement time history of Disp. 3 (lowest North Array displacement transducer).

Figure 4-15 shows the response to be a combination of linear and non-linear segments at and within different acceleration levels. Non-linear segments are visible in the 0.1g, 0.2g, 0.4g and 0.5g shaking steps (see Figure 4-14 above for complete 0.5g time history without scale break). For instance, during the 0.4g shaking step, a period of non-linear deformation is followed by a sudden increase in the rate of displacement and a linear response.

The non-linear segment could be due to the further development of failure surfaces already visible in Figure 4-4 (c), and the formation of new failure surfaces, within the backfill, whilst the sudden increase in displacement and a linear response could be the result of the weight of the wall face contributing to an increase in the overturning component of displacement, acting on the previously formed failure surface.

A similar combination of non-linear and linear displacement responses has been noted by Koseki et al. (2006) in reduced-scale shake-table tests on conventional gravity retaining wall models. In one test, displacement was recorded at the wall toe, and a non-linear displacement
response was found to occur for shaking levels prior to approximately 0.35g, before the formation of a failure plane within the backfill. This was attributed to non-linear shear deformation of subsoil layers below the wall. However after the formation of a failure plane within the backfill by 0.35g, displacement accumulated linearly. This was attributed to sliding at the wall base and subsoil interface.

This may help explain the reduction in residual displacement evident between shaking steps 0.3g and 0.4g. That is, progressive failure within the backfill means that the residual displacement accumulated during 0.3g shaking step could be associated with those shallow failure surfaces visible in Figure 4-4 (c), whereas residual displacement accumulated during 0.4g shaking step could be associated with failure surfaces deeper within the backfill, which are more difficult to develop. This would result in a reduction in the displacement recorded at the wall face.

In the final 0.5g shaking step shown in Figure 4-14, the wall deformed with large cyclic components of deformation and almost linear incremental residual (permanent) displacements until the test was terminated. A slight increase in the rate of deformation occurred just prior to the end of the final shaking step, most likely the result of the weight of the wall face as it tilted over, contributing to an increase in the rate of overturning. The test was terminated when the wall reached the maximum displacement of the displacement transducers and experimental set up, which occurred during the final few cycles of the 0.5g shaking step.

4.3.6 Investigation of irregular displacement response

Discernible in all tests at all shaking steps (except 0.5g where it appears “washed-out”) is a regular increased displacement response every third cycle, for instance that shown in Figure 4-12 and Figure 4-13, and hence occurs at a frequency of $5/3 = 1.67$ Hz. To investigate this further, raw facing displacement data is shown for 0.3g shaking step in Figure 4-16 below.
Figure 4-16. Facing and shake-table raw (unfiltered) displacement time histories at 0.3g shaking step.

The raw wall face displacement data plotted in Figure 4-16 shows an increased displacement at every 3rd cycle. However, the raw shake-table acceleration data shows two spikes at around 0.4g to 0.5g, for every one cycle hitting the desired 0.3g target acceleration. Additionally, the shake-table displacement does appear to be increased slightly every 3rd cycle.

Figure 4-17 (a) – (d) plots the fast fourier transform of the wall, reinforced soil and shake-table system to illustrate their respective frequency components and ascertain the origin of the 1.67 Hz component for the 0.3g shaking step. Figure 4-18 (a) – (d) shows the fast fourier transforms of the system when excited at 0.5g.
Figure 4-17. Fast fourier transforms for 0.3g shaking step for: (a) wall face top displacement, (b) acceleration at the top of the reinforced soil zone, (c) the base input acceleration (d) shake-table displacement.
Figure 4-18. Fast fourier transforms for 0.5g shaking step for: (a) wall face top displacement, (b) acceleration at the top of the reinforced soil zone, (c) the base input acceleration (d) shake-table displacement.
The main frequency components of the system as seen in Figure 4-17 and Figure 4-18 (a) – (d) are:

- The driving frequency of the input base motion at 5 Hz, and other modes at 15 Hz and higher.

- The fundamental natural frequency of the loaded shake-table at 13.7 Hz as determined previously in Section 3.5. No other natural modes are visible.

- Long-period (low frequency components) motion associated with the permanent displacement of the wall face visible in Figure 4-17 (a).

- A frequency spike at 3.35 Hz only visible in the wall face displacement and reinforced soil acceleration responses (Figure 4-17 (a) – (b) and Figure 4-18 (b)).

The latter point noted describes a frequency component exactly twice that deduced on inspection of the wall face displacement time histories. Further, it is no: visible in the input shake-table displacement and acceleration time histories, but appears within the reinforced soil zone and wall face (Figure 4-17 (a) – (b)). It is quite possibly "washed-out" within the shake-table displacement during 0.5g shaking.

One possible explanation is found upon comparison of the apparatus used to record the shake-table displacement and accelerometer time histories. The shake-table displacement is measured by a linear variable displacement transducer (LVDT) which is fixed to the shake-table foundation and records the relative shake-table top displacement. The base input acceleration is recorded on the shake-table top and, as a result, measures absolute accelerations (which includes shake-table and foundation movement).

Marriot (2009) recorded up to 1.1 mm of foundation movement during shaking table tests, and this too was noted in the current tests. This creates increased and irregular shake-table acceleration not recorded in the shake-table displacement time history. Thus it is considered that the response is likely to be the result of the hydraulic actuator driving the shake-table motion as opposed to model-specific behaviour.
4.4 Analysis of Test-6 results

This section develops seismic response measures for Test-6, to provide a general framework for the parametric study of L/H ratio and wall inclination influence on seismic performance in later sections.

4.4.1 Cumulative wall face displacement

The cumulative facing displacement for Test-6 is plotted as a function of the number of shaking cycles in sequence of increasing base input acceleration in Figure 4-19 (a) – (e). Note that each point is plotted at the displacement peak in the outwards (positive) direction and includes both cyclic and residual components of displacement. This is why the displacement recorded at the 50th cycle peak (50c) for 0.1g and 0.3g shaking steps is larger than the residual displacement recorded after shaking had stopped. This method ensures that the measurements recorded at each displacement transducer are time coincident; a displacement peak is better temporarily defined than a completed cycle (i.e. the baseline shifts due to residual displacement).

The residual displacement measured at the end of each shaking step was extrapolated to the wall top and toe at 900 mm and 0 mm respectively. Steel stiffeners attached to the front of the wall face ensured wall rigidity, and both vertical displacement transducer arrays recorded linear displacements, validating this extrapolation. The extrapolated values were then used to infer sliding and rotation components of deformation as discussed below.

Visible in greater detail is the predominance of overturning failure for all shaking steps, with only minor sliding recorded for shaking steps up to 0.5g. Further, upon application of 0.5g base input acceleration, most of the sliding component occurred during the first 30 cycles of shaking, which then decreased throughout the rest of the shaking step. As the wall rotates about the toe, it is likely that the weight of the wall face begins to play an increasing role in the wall response, and generates further rotation and prevents further sliding. In other words, as the wall overturns, the facing weight increases the destabilising moment causing further overturning. This coincides with a similar increase in the rate of displacement recognisable in the displacement-time history shown in Figure 4-14.
Figure 4-19. Cumulative lateral displacement of the wall face as a function of the base input acceleration and number of cycles for Test-6, reinforced at L/H = 0.6. NB: "e" denotes the peak for the number of cycles completed.
4.4.2 Overturning and sliding components of deformation

Displacements at the wall top and toe are determined from linear extrapolations in Figure 4-19 shown as dashed lines. The horizontal displacement measured at the wall top, $x_{\text{top}}$, is dominated by an overturning component ($x_{\text{rotation}}$) and includes some sliding component, $x_{\text{slide}}$, observed at the wall toe. Thus pure rotation of the wall is calculated as the difference between the wall top displacement, and wall toe displacements, $x_{\text{rotation}} = x_{\text{top}} - x_{\text{slide}}$.

Total, sliding and rotational components of residual displacement of the wall top at the end of each shaking step are plotted as a function of the input base acceleration in Figure 4-20. The acceleration for each shaking step is calculated as the average amplitude of the input base shaking using the double amplitude method.

![Figure 4-20. Rotation and sliding components of cumulative residual horizontal displacement of the wall top with increasing base input acceleration (PGA). Reinforced at $L/H = 0.60$ (T6).](image)

A number of comments may be made with respect to Figure 4-20. The total displacement-acceleration curve is bi-linear and typical of other physical model studies (El-Emam and Badhur 2004; Watanabe et al. 2003). At low base input accelerations, there is small
permanent deformation; at base input accelerations larger than some threshold value – in the above case, 0.4g – the rate of deformation increases significantly. As discussed previously, this value of acceleration could be defined as the critical acceleration for the model wall system and is shown as point A in Figure 4-20. A discussion on the selection of critical acceleration values, and their theoretical derivation is made in Section 2.2.5.

Prior to the critical acceleration being reached, the wall exhibits small sliding with some ~ 4 mm evident by the completion of 0.4g shaking step. Rotation contributes almost entirely to the total deformation of the wall top of ~ 33 mm. At shaking past the critical acceleration, sliding of the wall increases significantly and contributes ~ 27 mm to the total displacement of 195 mm, or about 13%.

The overturning failure mode is perhaps predominant due to a reduced confinement of the top layers of reinforcement, and thus a reduction in resistance to overturning. The low levels of sliding exhibited in Test-6 and other tests, could be the result of high localised friction between the FHR facing panel toe and the rigid foundation covered with a layer of glued sand. Hence the wall can only slide once this frictional limit is exceeded.

Interestingly, it is noted that even flexibly-faced models on a subsoil foundation (hence with low localised friction at the wall face), tested by Sabermahani et al. (2009) also exhibited small sliding components of deformation. Thus it is considered that the current experimental model does not inhibit the development of sliding deformation.

4.4.3 Critical Acceleration

Critical acceleration is defined by Bracegirdle (1980) as “the horizontal pseudo-static acceleration acting uniformly over the structure to achieve limiting equilibrium”. In other words, as the critical acceleration generates limit equilibrium (factor of safety equal to unity) any larger acceleration applied will cause the structure to slide.

Critical acceleration is important as a measure of stability; a high critical acceleration value indicates a high stability during an earthquake. Additionally, critical acceleration is also used for predictions of permanent displacement based on rigid block analysis (Richards and Elms 1979). Thus correct selection of a critical acceleration value for the structure under analysis, is one factor which helps the accuracy of such a prediction.
As noted above, the bi-linear displacement-acceleration curve in Figure 4-20 (a) demonstrates the existence of a threshold or critical acceleration; accelerations larger than 0.4g in Test-6 (point A) cause the wall displacement to increase significantly. Thus 0.4g could be considered the model specific critical acceleration value. However, determination of the critical acceleration for the other tests is not so clear. For instance, Section 4.6.3 shows Test-5 to have demonstrated an increase in the rate of wall top displacement both at 0.5g and 0.6g. Hence it is unclear as to what is the critical acceleration for Test-5.

Previous researchers have defined the critical acceleration observed in model studies with different measures. Watanabe et al. (2003) defined it as the acceleration coefficient that generated a cumulative displacement larger than 5% of the wall height (0.05H), or 25 mm, which was recorded at an elevation of 450 mm for the 500 mm high model walls used in their experiments. This definition was justified because wall top displacement increased rapidly after about 25 mm had been accumulated. Because, the models were subjected to amplitude-increasing irregular acceleration-time histories until wall failure, the critical acceleration criterion is based on displacement accumulated by previous shaking steps. This could be problematic as not a single acceleration coefficient can be defined that contributes to an increase in wall top deformation.

While the 0.05H value is specific to the Watanabe et al. (2003) models, the critical acceleration defined by this method is plotted as point B in Figure 4-20.

Nova-Roessig and Sitar (2006) defined critical acceleration as the minimum acceleration that caused the slope to deform permanently. Because permanent deformation occurred at accelerations as little as 0.03g, this was defined as the critical acceleration. With further shaking at higher accelerations, the critical acceleration value was observed to increase, and this was attributed to increased densification of the backfill which was observed. Hence the criterion does not provide a good indication as to when ultimate failure may occur.

For all of the present tests, permanent deformation occurred at the onset of the (initial) 0.1g base input acceleration and thus this would be defined as the critical acceleration (point C in Figure 4-20). However failure is actually observed at base accelerations far larger, and the definition yields little information as to when this failure might occur.

El-Emam and Bathurst (2005) compared rotation and sliding failure modes as a means of observing critical acceleration. The acceleration coefficient that caused a sudden increase in
sliding, compared to rotation, indicated the critical acceleration value. The method is more consistent with the Richard-Elms method (1979) of block sliding, for determining permanent displacements during an earthquake. In the Richard-Elms approach the critical acceleration is the acceleration coefficient that generates limiting equilibrium against sliding failure. Because critical acceleration is an important parameter for the prediction of permanent displacement, it is appropriate that the El-Emam and Bathurst (2005) criteria for observed critical acceleration is the same as its subsequent use for displacement predictions using block sliding.

The displacement data for Test-6 is broken into its components of rotation and sliding components, normalised by the height of the wall, and is replotted in Figure 4-21.

![Diagram](image)

Figure 4-21. Normalised sliding and rotation components of wall response for \( L/H = 0.6 \).

Figure 4-21 shows that this particular criterion is ill-suited to the Test-6 data set, as rotation is always dominant over sliding, even during ultimate failure. In contrast, the data presented by El-Emam and Bathurst (2004, 2005) demonstrated a distinct transition between rotation and sliding modes of deformation, most likely because the model wall's facing panel was founded on roller bearings of low localized friction in order to fully decouple measured vertical and horizontal toe load components. This boundary condition is unlikely to be representative of field structures.
Whilst no relative transition from rotation to sliding modes of failure was evident during Test-6, there was a significant increase in the magnitude of sliding prior to failure, and this perhaps, should form the basis of any measure of critical acceleration. This is further investigated with the comparison of all test results in Section 4.6.3.

### 4.4.4 Soil response

Fast Fourier transformations of the acceleration time histories of Test-6 for shaking steps 0.1g, 0.3g and 0.5g are shown in Figure 4-22.

![Fast Fourier Transformations](image)

**Figure 4-22.** Fast Fourier Transformations for the base input and response accelerations for the 0.1g, 0.3g and 0.5g base shaking levels for Test-6. The values in parentheses correspond to peak values.
The response accelerations have the same frequency content as the input base acceleration. This is a further verification of high soil density, as the full frequency content can be transmitted through the system without losses in frequency content associated with frictional damping etc. Conversely, a loose soil would act to dampen out high frequency components (Kramer 1996).

FFT magnitudes increase up the wall for all frequencies and are the result of amplification of the base acceleration. Further, this amplification increases with increasing base input acceleration. This is discussed further below.

4.4.5 Peak acceleration amplification

The average peak accelerations within the soil deposit are normalised with respect to the base input average peak acceleration and plotted in Figure 4-23. Only peaks in the outwards (positive) direction were used in the calculation as these are of primary concern and act to generate a larger destabilising force (accelerations acting inwards act to stabilise the wall).

![Figure 4-23. Amplification of peak outwards accelerations with wall elevation is plotted for each shaking step: (a) reinforced soil and (b) backfill.](image_url)

Figure 4-23 (a) demonstrates a significant amplification of the average peak base input acceleration in the reinforced zone up the wall for all shaking steps. This amplification increases with increasing base shaking from 0.1g to 0.5g; behaviour also observed by Matsuo et al. (1998) for flexible reinforcement. The base input acceleration near the wall top
(elevation of 815 mm) is amplified by 1.05 in the first shaking step of 0.1g, which then increases to around 1.2 – 1.3 for the 0.2g, 0.3g and 0.4g shaking steps. In the final shaking step of 0.5g, the soil response significantly amplifies the base input acceleration by 1.6. This is further evidence for some threshold acceleration value being reached at this acceleration level.

Figure 4-23 (a) also demonstrates that amplification in the reinforced zone is non-linear up the wall. This has also been reported by Nova-Roessig and Sitar (2006) in their dynamic centrifuge model tests. Ramifications of this for design are discussed in Section 4.7.1.

Figure 4-23 (b) shows a slight amplification of input base accelerations near the top of the ‘far-field’ backfill; however this is only a single measurement point, hence a dashed line is used to show amplification up the wall assuming a linear amplification response with height from the base of wall.

Larger amplification factors have been reported in the literature for both shaking table and dynamic geotechnical centrifuge tests (El-Emam and Bathurst 2004; Fairless 1989; Nova-Roessig and Sitar 2006). Fairless (1998) reports amplification factors of 3.0 at the top of 0.3 m high 1:6 model scale walls in tests where the acceleration input was pulsed. El-Emam and Bathurst (2004) report amplification factors of 2.25 at the top of 1 m high walls constructed with D_r = 84% for shaking table tests just prior to failure, and a decrease in amplification during the final shaking step to 2.0.

Nova-Roessig and Sitar (2006) also reported amplification at wall top of peak acceleration values of up to 2.3; however this occurred at low base input accelerations up to 0.15g. The authors noted that, amplification in general was accompanied by some shearing. In direct contrast to Figure 4-23, the authors reported that de-amplification (attenuation) of peak acceleration occurred at base accelerations larger than about 0.46g.

The difference in behaviour reported by Nova-Roessig and Sitar (2006) and that shown in Figure 4-23 (a) could be contributed to by two effects: the difference in initial model densities and the difference in frequency contents of the input motions used. Nova-Roessig and Sitar (2006) constructed their walls at a D_r = 55% and used multiple sinusoidal motions at low amplitudes to examine elastic behaviour, then a series of 8 to 12 time-histories of wide frequency content, amplitude-scaled to 0.11g to shake each model. This is compared to a stepped-sinusoid of predominant frequency 5 Hz used in the present tests.
The peak response accelerations recorded during higher amplitude shaking by Nova-Roessig and Sitar (2006) are typically high frequency components, and these would be dampened out with the accompanying higher rates of shearing deformation to generate deamplification. This did not occur in the present tests because the peak accelerations all occurred at a low frequency of 5 Hz.

Additionally, at low accelerations there is little non-linear soil behaviour (limited damping) and amplification is in general, greater for looser soils (Kramer, 1996). High amplification at low acceleration levels was not evident in the present tests because the initial density of the model backfill was already high ($D_r = 90\%$).

### 4.4.6 RMS acceleration amplification

Differences in amplification factors between the results can also be attributed to the choice of acceleration measure used to calculate amplification factors. Both El-Emam and Bathurst (2004) and Nova-Roessig and Sitar (2006) reported amplification factors based on peak outwards accelerations. For design purposes this could be misleading since the high frequency peak values occur for a short duration and are rarely important for design purposes. Moreover, as discussed previously, a single parameter, such as Peak Ground Acceleration (PGA) is insufficient to completely describe an earthquake's damage-causing ability.

Thus a number of measures exist that attempt to combine frequency, duration and amplitude to adequately quantify an acceleration time history. These include the Root Mean Squared (RMS) acceleration, Acceleration Power and Arias Intensity, measures that include the duration, acceleration amplitude and frequency content (somewhat) of acceleration-time histories. The first of these, RMS acceleration is defined in Equation 4-2.

\[
RMS = \left( \frac{1}{T} \int_0^T a(t)^2 \, dt \right)^{\frac{1}{2}}
\]  

(4-2)

Where RMS is the root mean square acceleration, $a(t)$ is the acceleration time history, and $T$ is the duration of shaking. Following the method used by Law and Ko (1995), the acceleration time histories were converted to RMS acceleration values. Each RMS response was then normalised with the base input RMS acceleration to generate amplification factors as in Equation 4-3.
\[ AF = \frac{RMS(h)}{RMS(base)} \]  

(4-3)

Where AF is the amplification factor, and RMS(base) and RMS(h) are the root mean squared accelerations at the base and some height, h respectively. RMS acceleration thus quantifies the entire time history (not just positive peak accelerations). The acceleration response data is reinterpreted and RMS based amplification factors are plotted with elevation for the reinforced zone and backfill in Figure 4-24.

![Graph showing amplification of RMS acceleration for reinforced zone and backfill soil](image)

Figure 4-24. Amplification of RMS acceleration up the wall for each shaking step: (a) Reinforced zone and (b) Backfill.

Figure 4-24 shows similar non-linear behaviour of the reinforced soil zone and larger amplification with increasing acceleration amplitude. However amplification factors are smaller than those calculated for peak acceleration. For instance, the RMS amplification factor calculated at 0.5g near the top of the reinforced soil zone is 1.42 compared with a peak amplification of 1.62. This confirms that the peak acceleration amplification can over estimate the acceleration response of the wall overall.

### 4.4.7 Acceleration amplification during shaking

Amplification of acceleration at each shaking step as discussed above is an average response for the entire duration of shaking. We know that the facing displacement response is non-linear during shaking, thus it could be expected that the soil response would also vary during
each shaking step. Figure 4-25 shows the amplification factors at the beginning, mid-way and end of shaking steps 0.1g, 0.3g and 0.5g. The amplification factors are calculated from RMS acceleration values for 5 cycle periods at 0 - 5 cycles, 20 - 25 cycles, and 45 - 50 cycles for each shaking step.

Figure 4-25. Acceleration amplification factors for a 5 cycle period starting at cycles 0, 20c and 45c for shaking steps a) 0.1g, b) 0.3g and c) 0.5g.
Similar to the wall face displacement response during shaking, Figure 4-25 shows that the soil response varies during each shaking step. For instance, as expected, the 0.1g shaking step exhibits a larger amplification factor during the first 5 cycles than at later stages in the shaking step. This response is subsequently reduced as the reinforcement becomes engaged and acts to stiffen the wall.

At later shaking steps of 0.3g and 0.5g, the soil response is more consistent during shaking with similar amplification values recorded. The increased amplification factors recorded in the last 5 cycles of 0.5g shaking is the result of the wall face reaching the physical limit of the box and hitting the brackets supporting the displacement transducer arrays.

4.5 Parametric study

4.5.1 Detailed summary of testing series

During testing, the experimental parametric study investigated the variation of the L/H ratio and wall inclination on seismic performance of the GRS wall models. A summary of the parameters varied for each test, and the issues encountered during testing was presented in Table 4-1.

As stated previously in Section 3.3.3, the seal between the model wall facing panel and the side of box had to achieve two aims: 1) Minimise boundary effects with the lowest possible friction between the wall facing panel and the box sidewall interface, and 2) prevent leakage of sand around the facing panel. The seal design was investigated during the testing process, and as a result, Tests-1, 2, 3 and 4 seals varied from that presented in Section 3.3.3. Where the seal was observed to influence model behaviour, the data was not used. This resulted in Test-1 data pre-failure, and all of Test-4 data to be considered unreliable and not used.

In Test-1, cellotape was used to seal the wall, as the designed seal did not prevent sand from leaking around the side of the wall. The cellotape was observed to impact on wall response at low acceleration levels, restricting free movement. However during the final shaking step, the cellotape failed in a brittle manner and allowed free movement of the wall, and this data during failure was considered reliable.

In Test-2, again the seal did not prevent the leakage of sand between the wall face and box sidewall, and some small quantity of sand was lost prior to testing and during testing. In
order to prevent a further loss of sand, testing was initiated at 0.3g, and continued as per normal procedures. In the 15 minutes between testing stages (necessary to download image data), foam was placed along the side of the panel to prevent further sand leakage. This was removed prior to shaking again. The behaviour during these later stages was similar to behaviour observed for Test-6 (same L/H ratio and geometry), and this would indicate the seal did not significantly affect model behaviour in Test-2.

In Test-3, the model was subjected to a 10 Hz excitation (theoretical PGA of 0.4g) due to testing error, however rather than being a weak point in the testing, this was used as an opportunity to compare the effect of frequency on model response. Because the frequency was higher, an 8th order 15 Hz low pass Butterworth filter was used for the time history plots presented in Appendix B. This shows, however, that the model was subjected to an actual acceleration amplitude of approximately 0.6g.

For Test-4, (a repetition of Test-1 reinforced at L/H = 0.75) the facing seal had high friction, evident in the test’s time histories presented in Appendix B. This high friction resulted in a reduced facing displacement than could be expected upon comparison with the other tests, and the wall failed at a higher PGA of 0.65g, as opposed Test-1 which failed at 0.6g. Hence it was deemed that all data obtained during Test-4 was impaired and was not used in the study.

For Tests-5, 6, and 7 the seal as described in Section 3.3.3 was used. The seal’s friction was tested and was determined to be low (friction force against rotation ~ 1 N), and no sand leaked during testing. Of greater importance were the results generated; these were deemed reliable in general, with parameter variation causing a marked effect on model behaviour which is discussed subsequently.

A comment on the repeatability of the testing apparatus and consistency of the construction methodology is made in Section 4.5.3, with reference to Tests-2 and 6, both vertical walls reinforced at L/H = 0.6. While Test-2 and 6 models were ‘sealed’ differently, the behaviour at later stages of Test-2 was somewhat similar to Test-6, even though testing was initiated at 0.3g due to sand leakage in Test-2.

The model geometry for all tests is presented in Section 3.4.8. The acceleration and facing displacement time-histories for all tests are presented in Appendix B. A study of the effect of L/H ratio and wall inclination is made in the following sections.
4.5.2 Displacement-acceleration curves

The displacement-acceleration curves for Tests-1, 5, 6, and 7 are plotted in Figure 4-26. As noted above, Tests-2, 3, and 4 are not plotted due to issues surrounding the seal and testing procedure.

![Graph showing displacement-acceleration curves](image)

Figure 4-26. Displacement-acceleration curves for Tests-1, 5, 6, and 7. Note that Test-1 data prior to 0.5g is not plotted, for reasons as discussed above.

As can be seen in Figure 4-26, residual displacement was steadily accumulated with each increasing shaking step until some critical acceleration value at which displacement increased markedly, and this confirms the generally bi-linear behaviour expected for GRS models (Watanabe et al. 2003, El-Emam and Bathurst 2004). The tests also demonstrated behaviour consistent with what could be expected for the variation of L/H ratio and wall inclination tested. Specifically, an increase in L/H ratio from 0.6 to 0.9, and a reduction in
inclination of the wall from vertical to 70° with the horizontal, resulted in the displacement-acceleration curve to be shallower (lower residual displacement for increasing acceleration shaking), and an increase in the acceleration required to cause failure.

Failure was defined by a significant increase in rate of wall top displacement, which for the current tests resulted in the limit of the experimental apparatus to be reached. Figure 4-26 shows failure occurred at acceleration shaking levels of 0.5g, 0.6g and 0.7g depending on the L/H ratio for Tests-6, 1, and 5 respectively. Test-7 (the inclined wall) also failed at 0.7g. The increase in acceleration required to cause ultimate failure demonstrates the increase in stability under seismic conditions gained due to the choice of reinforcement and/or wall geometry parameters.

It can be seen in Figure 4-26 that for all tests, the residual displacement accumulated prior to failure at low acceleration levels, was non-linear. In other words, a decrease in incremental displacement (the displacement accumulated within each shaking step) is observed prior to the critical acceleration being reached. This was noted in the facing displacement time-histories of Test-6 and was briefly described in Section 4.3.5. This effect is most marked in Test-5: Except for the 0.1g shaking step, incremental displacement decreased up to 0.5g, when incremental displacement increased suddenly for shaking steps 0.6g and 0.7g (wall failure). This behaviour was not observed in the tests by Watanabe et al. (2003) or El-Emam and Bathurst (2004; 2007).

With reference to the facing displacement time-histories in Appendix B, there are segments of non-linear and linear accumulation of displacement. As stated in Section 4.3.5, this is most likely due to the mode of failure and the progressive nature in the development of failure surfaces within the backfill. The development of deformation within the backfill is discussed further in Chapter 5.

The effect of parameter variation on the rate of residual displacement accumulation, mode of failure, and measures of stability such as critical acceleration and acceleration amplification is presented in detail in Section 4.6.

4.5.3 Repeatability of experimental results

Two vertical walls reinforced at L/H = 0.6 (Tests-2 and 6) were constructed and tested and provide information as to the reliability of the model construction, experimental method, and
results. Test-2 leaked some sand around the wall face and in order to minimise this, shaking was initiated at an acceleration input of 0.3g instead of the usual 0.1g as for Test-6. Except for the seal, both walls were constructed with the same technique and target relative density. It could be surmised that as sand leaked during Test-2, the friction at the facing panel and side of the box was minimal, and hence similar behaviour was expected for both tests.

![Graph](image)

**Figure 4-27.** Comparison of displacement-acceleration curves for repeatability purposes of Test-2 and 6 both reinforced at L/H = 0.6.

Figure 4-27 shows good agreement between the Tests-2 and 6, especially at 0.4g and 0.5g shaking steps. Test-2 exhibited less initial sliding, thus the wall top was able to rotate more and register the higher lateral displacement of 205 mm compared to 193 mm during the final shaking step at 0.5g.

For Test-6, the incremental displacement accumulated during the 0.3g shaking step is similar to that recorded for the single 0.3g shaking step in Test-2, even though Test-6 had previously undergone two previous shaking steps at 0.1g and 0.2g. Koseki et al. (1998) notes that for scale model tests, the incremental increase in base input acceleration should be large enough to reduce the possible effects of previous shaking history on subsequent model response. Koseki et al. (1998) assumed that 0.05g increments were sufficient to achieve this; given that similar incremental displacements were recorded during the 0.3g shaking step for both Test-2
and 6, it appears that a 0.1g increment is sufficient in the current tests to reduce the effect of previous shaking history on model response.

4.6 Influence of L/H ratio and wall inclination on seismic performance

4.6.1 Accumulation of residual displacement

Residual displacement was accumulated with each shaking step and its rate of accumulation varied according to reinforcement L/H ratio and model geometry. This is discussed below.

_Influence of L/H ratio on accumulation of residual displacement_  

Reinforcement L/H ratio was varied across Tests-6, 1, and 5 reinforced by L/H = 0.6, 0.75 and 0.9 respectively to determine the parameter's influence on seismic performance. Figure 4-28 shows the displacement-acceleration curves for the residual displacement at the top of the wall for these three tests.

Figure 4-28 clearly shows the stabilising effect of increasing the reinforcement L/H ratio, for the range of L/H tested. For instance, failure occurred at 0.5g, 0.6g and 0.7g for L/H = 0.6, 0.75 and 0.9, respectively.

Whilst no reliable data pre-failure was recorded for Test-1 reinforced at L/H = 0.75 (for reasons discussed above), pre-failure displacement decreased with increasing reinforcement length via increasing the L/H ratio. For instance, a 50% increase in the L/H ratio from 0.6 to 0.9, caused a 33% decrease in the cumulative lateral displacement recorded up to 0.3g base input acceleration. Similar decreases in lateral facing displacement with increasing L/H ratio have been recorded in reduced-scale GRS wall model tests by Matsuo et al. (1998) and El-Emam and Bathurst (2004).
Figure 4.28. Comparison of displacement-acceleration curves for tests of different L/H ratio. NB: Test-1 is dashed to represent unreliable data.

Whilst a somewhat linear relationship between increasing the L/H ratio and an increase in the critical acceleration (and ultimate failure) can be seen, this is not clear. For instance, Sakaguchi (1996) showed a diminishing return in the reduction of residual displacements for L/H ratios larger than 0.67.

A definitive relationship between an increase in the L/H ratio being met with a proportional increase in seismic performance is unable to be inferred because: 1) the base input acceleration increment is reasonably coarse (increments of 0.1g), 2) both tests reinforced at L/H = 0.75 had some sealing issues, and 3) the range of L/H ratios (0.6 – 0.9) tested was only small.

Somewhat contrasting the observed increase in seismic performance with an increase in the L/H ratio from 0.75 to 0.9, is centrifuge work by Nova-Roessig and Sitar (2006). In their
testing, only a minimal reduction in lateral facing displacements was observed when the L/H was increased from L/H = 0.7 to 0.9. However, this range in parameter values is smaller than the current tests, and the absence of a more marked trend could be due to a variety of factors such as different reinforcement spacing, wall density, and the wrap-around facing all perhaps acting to reduce the impact of reinforcement length on lateral residual displacements.

Of interest is the behaviour observed just prior to failure. Test-6 reinforced the shortest at L/H = 0.6 failed abruptly after the critical acceleration of 0.4g was reached. In contrast, Test-5 reinforced the longest at L/H = 0.9, displayed a somewhat 'transitional' region at 0.6g, where displacements markedly increased, however failure did not occur until 0.7g.

This could possibly be attributed to the deformation of the Test-5 reinforced soil block. For instance, the reinforced soil zone of Test-6 behaved more like a rigid block, with minimal internal deformation as compared to the reinforced soil block of Test-5 where larger internal deformation was observed. Hence it was more difficult for the larger reinforced soil block to overturn and slide as a block and this could have reduced the lateral displacement recorded at larger accelerations. This is further discussed in Chapter 5.

*Influence of wall inclination on accumulation of residual displacement*

The displacement-acceleration curves of Test-1, 5 and 7 are shown in Figure 4-29. Test-1 is included because it was reinforced with the same reinforcement layout of L/H = 0.75 as Test-7, however deformation data prior to failure is unreliable due to seal issues. Instead, as a pre-failure comparison, displacement data of Test-5, reinforced at a larger L/H = 0.9, is plotted.
Figure 4-29. Comparison of displacement-acceleration curves for the effect of wall face inclination on stability.

As expected, comparison of the final shaking levels of Tests-1 and 7, Figure 4-29 shows that inclining the wall face acts to increase stability such that failure occurs at 0.6g for the vertical wall and at 0.7g for the wall inclined at 70° to the horizontal. However, in terms of wall top displacement accumulated during low acceleration pre-failure, the effect of wall inclination is almost the same as increasing the reinforcement up to L/H = 0.9 for a vertical wall face. A direct comparison of the effect of the wall inclination cannot be made in the current tests, given that the Test-1 data is unreliable. However, with reference to Figure 4-26, a median response between Tests-5 and 6 reinforced above and below L/H = 0.75 at L/H = 0.9 and 0.6 respectively, can be inferred as indicative of behaviour for a vertical wall reinforced at L/H = 0.75. This would show that there could have been a significant reduction in facing displacements due solely to wall inclination.

At the same base input acceleration of 0.6g just prior to failure, the inclined wall showed a slightly lower displacement response at the top of the wall than the vertical wall reinforced at
a higher L/H = 0.9. This indicates that an inclined wall is better able to resist displacement at higher acceleration shaking than a vertical wall.

Similar results have been reported by Matsuo et al., (1998) and El-Emam and Bathurst (2005) with wall inclinations ranging from 79° and 80° to the horizontal, respectively. In these tests, a shallower displacement-acceleration curve, compared to the vertical walls, was observed.

For instance, El-Emam and Bathurst (2005) show that at the vertical wall’s critical acceleration of 0.36g, the inclined wall at 80° to the horizontal, accumulated approximately only 50% of the residual displacement as the vertical wall. A similar reduction was recorded for Test-7 inclined 10° further to 70° to the horizontal. Thus it is possible there is a diminishing benefit to wall inclination less at 80° to the horizontal; however this requires further investigation.

4.6.2 Modes of failure

As noted previously, the walls failed predominantly by overturning and sliding failures. However, lateral displacement recorded at the wall top is a function of both rotation of the wall and sliding of the wall toe; it is convenient to separate these two modes of failure.

Figure 4-30 (a – c) plots the components of rotation and sliding during testing of Tests-5, 6 and 7 reinforced at L/H = 0.6, 0.9, 0.75, respectively (a – c, respectively). The first two were vertical walls, while Test-7 was inclined at 70° to the horizontal. As mentioned previously, no reliable data at low levels of base input acceleration is available for Test-1, reinforced at L/H = 0.75, and no comparison is made. It is important to note that the dashed lines representing the rotation and sliding components have been calculated, as discussed previously in Section 4.4.2.
Figure 4-30. Comparison of failure modes during Tests-5, 6, and 7 for walls reinforced at L/H = 0.6, 0.9 and 0.75 inclined at 70° to the horizontal. The letters represent the different definitions of critical acceleration proposed by various researchers and are discussed further in Section 4.6.3.

It can be seen that for all tests, the predominant component of failure was by rotation of the wall top, as opposed to sliding, which was minimal. Sliding only became significant during the final shaking step for each wall.

The effect of the L/H ratio and wall inclination on different modes of failure is discussed below.
Effect of L/H ratio on components of failure

For both vertical walls, the L/H ratio had a minimal effect on the components of failure. That is, despite a 50% increase in reinforcement length from L/H = 0.6 to 0.9, failure was predominantly rotational (Figure 4-30 (a and b)). Significant sliding only occurred during the final shaking step of each wall at failure.

However, an increase in the L/H ratio (Test-5) decreased the sliding component of deformation when compared to Test-6 pre-failure. This further highlights the greater stabilising effect of a larger L/H ratio pre-failure.

Effect of wall inclination on components of failure

Similar to the vertical walls, total wall top deformation largely comprised rotation, with only some small sliding which occurred pre-failure (Figure 4-30 (c)). The cumulative displacement of Test-7 during testing is replotted in Figure 4-31 to better illustrate the geometry of deformation.
Figure 4-31. Test-7 cumulative displacement recorded at the wall face showing the geometry of failure from the start of testing (EoC), through to 0.7g.

Figure 4-31 shows that significant sliding occurred only during the final 0.7g shaking step. However whilst the predominant mode was rotation, the Test-7 sliding component of 3.9 mm pre-failure was almost twice that of Test-5 pre-failure of 2.0 mm. Further, at failure, sliding was a major component of failure and was nearly 40% of the total displacement recorded at the wall top. In comparison, the total displacement at the wall top recorded at failure for Test-5 comprised only approximately 30% sliding.

Figure 4-31 shows that because the wall was inclined, there was no increase in the overturning moment of the wall contributing to increased deformation. Instead, sliding was emphasised.

**4.6.3 Critical Acceleration**

The critical acceleration was defined previously by various researchers in Section 4.4.3 and indicated in the plots of Figure 4-30 above. As seen in Figure 4-30 (a), point ‘A’ describes a
typical definition of the critical acceleration of a sudden change in the rate of deformation. For the Test-6 displacement data, the criterion is clear. However, as noted in Section 4.4.3 above, and seen in Figure 4-30 (b) and (c), the criteria are not clear as to what the appropriate critical acceleration should be, as there is some sort of ‘transition’ region between behaviour at low shaking levels and high shaking levels. Thus the original criterion could be used to define either points ‘B’ or ‘C’.

The El-Emam and Bathurst (2004, 2005) definition of critical acceleration (described in Section 4.4.3) is a relative change between rotation and sliding components. This is plotted Figure 4-32 for both Test-6 and 5, reinforced at L/H = 0.6 and 0.9, respectively (components normalised by wall height, H).

![Graph showing normalized wall rotation, x(top-xslide)/H vs normalized wall sliding, xslide/H](image)

**Figure 4-32.** Comparison of critical acceleration of Tests-7 and 5 reinforced at L/H = 0.75 and 0.9; Test-7 with facing inclined at 70 deg.

As shown in Figure 4-32, the prescribed criterion seems to be ill-suited to the present dataset because all walls demonstrated predominantly rotational behaviour.

Instead, because sliding only became significant upon failure, it is proposed that a sudden increase in the rate of sliding is sufficient to describe the critical acceleration of the wall. This is point D plotted in Figure 4-30 (a - c) above.
The sliding component of deformation for Tests-5, 6 and 7 is shown in Figure 4-33. For all tests, there is a significant increase in the magnitude of sliding just prior to failure, and this could be used to indicate the critical acceleration of the model wall. This threshold is exhibited for both vertical and inclined walls and for L/H ratios of 0.6 to 0.9.

![Graph showing sliding displacement vs. input base acceleration](image)

**Figure 4-33. Comparison of sliding components of deformation for Tests-5, 6 and 7.**

It should be noted that the apparent decrease in cumulative sliding displacement of Test-7 from 3.9 mm (at 0.2g) to 3 mm (at 0.6g) is due to the extrapolation used to determine the displacement at the toe of the wall face. Rather, it is likely that the wall slid until the toe “dug in” at 3.9 mm and the facing panel was subjected to some small deformation (of approximately 0.9 mm) and was not exactly linear as previously assumed. However, this does not have an effect on the overall observation that sliding becomes significant at failure.

Sliding is only one possible mode of external instability considered in design codes. For performance based design, some displacement predictions (Richards and Elms 1979) are
based on limit equilibrium against sliding, and thus should use critical acceleration as determined above. The above evidence would seem to validate the use of critical acceleration based on sliding as opposed to any of the other critical acceleration measures proposed above. The possible reasons for sliding deformation controlling the onset of global failure are discussed in Chapter 5.

4.6.4 RMS acceleration amplification

Acceleration amplification results in larger destabilising forces acting on the wall. Hence it is important to determine whether the parameters under investigation contribute to a reduction in the acceleration amplification observed.

Effect of L/H ratio on acceleration amplification

Section 4.4.6 and Figure 4-24 above demonstrated that the amplification of acceleration was non-linear up the vertical wall for Test-6. Figure 4-34 compares amplification factors of the top most accelerometer in the reinforced zones and backfill, Acc 4 and Acc 5, respectively, during all stages of testing for Tests-5 and 6 (reinforced with L/H = 0.9 and 0.6 respectively). It should be noted that the filtered and corrected acceleration time-histories were used in the calculations.
Figure 4-34. Comparison of RMS Amplification factors measured near the wall top within the reinforced zone (Acc 4) and backfill (Acc 5) for Tests-5 and 6 reinforced at L/H = 0.9 and 0.6 respectively.

Figure 4-34 shows both tests to demonstrate larger amplification to occur in the reinforced soil than the backfill zone, and exhibit a general trend of increasing amplification with increasing base input acceleration, with the highest amplification factors of 1.43 and 1.38 occurring during the final shaking steps of 0.5g and 0.7g at failure for Tests-6 and 5 respectively.

As noted in Section 4.4.5, these values are slightly lower than recorded by El-Emam and Bathurst (2007), and significantly lower than those recorded as 3.0 by Fairless (1989) and 2.3 by Nova-Roessig and Sitar (2006). This reflects differences in the experimental model as highlighted above.

Up to 0.2g, the amplification response for both walls is similar. At 0.3g, Test-5 reinforced at a larger L/H = 0.9, demonstrated a decreased amplification response until 0.5g, while Test-6 continued to rise until 0.4g and subsequent failure at 0.5g. Test-5 then demonstrated increased amplification again until failure at 0.7g. The decreased amplification response
exhibited by Test-5 when compared to Test-6 is evident for all shaking levels except 0.1g and could be attributed to its increased stability due to a 50% increase in L/H ratio. A similar decrease in amplifications observed with increasing L/H has been observed by El-Emam and Bathurst (2007).

A sudden increase in backfill amplification recorded by Acc 5 for Test-5 at base input accelerations larger than 0.5g can be attributed to a failure plane which extends from the back of the reinforced soil block and daylights at some distance beyond Acc 5. Hence during 0.6g and 0.7g shaking, Acc 5 records acceleration behaviour attributable to the active wedge, rather than “far-field” conditions. This is further discussed in Chapter 5.

**Effect of wall inclination on acceleration amplification**

Figure 4-35 plots the amplification of acceleration within the reinforced zones and backfill, during all stages of testing for Tests-7. It can be seen that amplification of acceleration is non-linear up the wall as found for the vertical walls.

![Figure 4-35](image)

**Figure 4-35. RMS acceleration amplification up the wall during Test-7 for:**
- a) Reinforced soil vertical accelerometer array (Acc 2, 3, 4) and b) Accelerometers Acc 1 and 5 where a linear relationship has been assumed.

Figure 4-36 compares the RMS amplification at the top of the wall for Tests-5 and 7 to determine the effect of wall inclination on acceleration amplification.
Figure 4-36. Comparison of RMS Amplification factors for Tests-5 and 7 reinforced at $L/H = 0.9$ and $0.75$; Test-7 inclined facing at 70°.

Up to 0.5g, the figure shows a similar amplification response for both walls with the inclined wall demonstrating slightly higher amplification. At 0.6g, Test-5 (vertical), shows an increased amplification while Test-7 (inclined) amplifications remained steady until 0.7g and failure. The reduced amplification response of Test-7 compared to Test-5 is a result of increased stability of the inclined wall compared to the vertical wall, even though the vertical wall was reinforced longer at $L/H = 0.9$ compared to Test-7 reinforced at only $L/H = 0.75$.

Figure 4-34 and Figure 4-36 show that generally, amplifications increase for all base input acceleration levels and for all tests, regardless of $L/H$ ratio and wall inclination. Contrasting these results is amplification data for shake-table tests by Matsuo et al. (1998) which showed amplification to be large within the potential sliding block at low acceleration levels, and attenuation to occur once noticeable plastic deformation, i.e. a failure surface has formed. Attenuation of large accelerations was also found in centrifuge tests by Nova-Roessig and Sitar (2006).