EXECUTIVE SUMMARY

OVERVIEW
Geosynthetic reinforced soil (GRS) walls involve the use of geosynthetic reinforcement (polymer material) within the retained backfill, forming a reinforced soil block. The geosynthetic reinforcement acts to transmit the overturning and sliding forces on the wall to the backfill through the tensile capacity and frictional resistance of the reinforcement. Key advantages of GRS systems include the reduced need for large foundations, cost reduction (up to 50%), lower environmental costs and faster construction. Furthermore, significantly improved seismic performance of GRS structures compared to conventional retaining structures has been observed in previous earthquakes such as Northridge (1994), Kobe (1995) and Chi-Chi (1999).

Design methods in New Zealand have not been well established with several different overseas standards and design guidelines currently used to design GRS structures in New Zealand. As a result, GRS structures do not have a uniform level of seismic and static resistance; hence involve different risks of failure. Although good seismic performance of GRS structures has been observed, this is likely due to their underlying stability under high seismic loads and conservatism in static design. Further research is required to better understand the seismic behaviour of GRS structures to advance design practices. Full-height rigid (FHR) facing type walls with its greater seismic performance are focused on in this study as it has become standard retaining wall construction technology for railways including bullet trains in Japan which has a similar seismic risk to New Zealand.

EXPERIMENTAL STUDY
The experimental study of this research involved a series of twelve 1-g shake table tests on reduced-scale (1:5) GRS wall models using the University of Canterbury shake-table. The seismic excitation of the models was unidirectional sinusoidal input motion with a predominant frequency of 5Hz and 10s duration. Seismic excitation of the model commenced at an acceleration amplitude level of 0.1g and was incrementally increased by 0.1g in subsequent excitation levels up to failure (excessive displacement of the wall panel). The wall models were 900mm high with a full-height rigid facing panel and five layers of Microgird reinforcement (reinforcement spacing of 150mm). The wall panel toe was founded on a rigid foundation and was free to slide. The backfill deposit was constructed from dry Albany sand to a backfill relative density, Dr = 85% or 50%. Densification of the backfill was achieved through a combination of a 950kg compactor plate and model vibration.

The influence of GRS wall parameters such as reinforcement length and layout, backfill density and application of a 3kPa surcharge on the backfill surface was investigated in the testing sequence. Uniform reinforcement length ratio (L/H) was varied between 0.75 and 0.9. The staggered reinforcement layout models had longer reinforcements for the top two layers (L/H = 0.9) with the rest of the reinforcement layers at a uniform length ratio, L/H of 0.75. Backfill densities were varied between Dr = 85% and 50% with majority of the wall models constructed at Dr = 50%.

Through extensive instrumentation of the wall models, the wall facing displacements, backfill accelerations, earth pressures and reinforcement loads were recorded at the varying levels of model
excitation. Additionally, backfill deformation was also measured through high-speed imaging and Geotechnical Particle Image Velocimetry (GeoPIV) analysis. The GeoPIV analysis enabled the identification of the evolution of shear strains and volumetric strains within the backfill at low strain levels before failure of the wall. This allowed interpretations to be made regarding the locations of dominant strain development (strain localisation) and progression of shear bands within the retained backfill.

**KEY FINDINGS**

For all wall models, rotation of the wall about its toe was the predominant failure mechanism with sliding only significant in the last two excitation levels (0.4 – 0.5g for staggered reinforcement models and L/H=0.75 models or 0.5g – 0.6g for L/H=0.9 models) with the greatest incremental increase observed at failure. A bi-linear displacement acceleration curve was observed with the existence of a critical acceleration level beyond which the rate of displacement increases sharply resulting in failure. In general, increase in acceleration amplification with increasing excitation was observed with amplification factors of up to 1.5 recorded in the reinforced backfill region. Maximum seismic and static horizontal earth pressures were recorded at failure and were recorded at the wall toe. The lowest reinforcement layer (deepest in the backfill) recorded the highest reinforcement load among the three instrumented reinforcement layers with the peak load occurring in the excitation level before failure. Gradual development of reinforcement load with the peak reinforcement load achieved at failure was observed for the top reinforcement layer.

For the staggered reinforcement layout, the increase of reinforcement length for the top two layers by 20% led to a reduced wall displacement of about 20% recorded at the excitation level prior to failure. Influence of the longer top two reinforcement lengths on critical accelerations were unable to be identified due to the coarseness of excitation level increments of 0.1g. The extended top reinforcement lengths restricted the rotational component of displacement during excitation, resulting in a greater sliding component contribution to wall displacement at failure. Lower amplification factors were observed for the longer uniform reinforcement length models due to reduced model deformation. Longer reinforcement lengths were observed to result in greater reinforcement loads developed within the reinforcement layer. A greater distribution of reinforcement load towards the top two extended reinforcement layers was observed in the staggered wall models.

An increase of model backfill relative density from Dr = 50% to Dr = 85% was found to induce greater stability of the wall (thus increasing its critical acceleration) than an increase of uniform reinforcement length by 20% with shallower displacement-acceleration curves observed. Minimal influence of backfill density on the wall failure modes were observed with further tests advised to verify this observation. The looser backfill models experienced greater acceleration amplification due to lower stiffness of the model. Greater reinforcement loads were developed in the higher density wall models due to greater confinement of the reinforcement layers. Similarly, less wall movement was required to engage the reinforcement layers and mobilise the reinforcement resistance compared to the lower density wall models.

The application of surcharge on the backfill was observed to initially increase the wall stability due to greater normal stresses within the backfill. However at greater excitation levels, the surcharge contribution to wall destabilising inertial forces outweighs its contribution to wall stability resulting
in wall failure occurring at the same excitation level as the non-surcharged models. As a result, no clear influence of surcharge on the critical acceleration of the wall models was observed. The surcharge acts as a damper during excitation, as a result, lower acceleration amplification factors were observed for the surcharged models. The application of the surcharge also increases the magnitude of reinforcement load developed due to greater confinement and increased wall destabilising forces.

The rotation of the wall panel resulted in the deformation of the backfill in the form of inclined shear surfaces that extended from the backfill surface to the ends of the reinforcement (edge of the reinforced soil block). These shear surfaces are observed to develop progressively with depth within the backfill with increasing excitation level, starting from the top of the backfill towards the toe of the wall. Development of vertical shear bands extending along the vertical edge of the reinforced soil block was observed in the higher levels of excitation, close to the failure excitation level. The resultant failure plane extended from the backfill surface to the lowest reinforcement layer, extending along its length before connecting with the toe of the wall. This is confirmed through development of well-defined failure planes that extended from the backfill surface to the lowest reinforcement layer and at the wall toe was observed at the critical acceleration point (the excitation level prior to failure). The resultant failure plane would have formed a two-wedge failure mechanism.

Key observations of the effect of different wall parameters from the GeoPIV results are found to be in good agreement with conclusions developed from the other forms of instrumentation. In the staggered reinforcement model, the longer top two reinforcement layers prevented the development of a preliminary shear band at the middle reinforcement layer, providing increased shear resistance within the backfill in the initial stages of excitation which would have contributed positively to the wall stability. A higher backfill density resulted in relatively low strain development (backfill deformation) and greater dilative strains observed within the backfill. The initial stabilising effect of the surcharge and destabilising effect in the later stages of excitation was also observed in backfill strain development.

**CONCLUSION**

This study has investigated the influence of reinforcement length and layout, backfill density and surcharging on the seismic performance of GRS walls but further research is required to achieve the goal of developing seismic guidelines for GRS walls in geotechnical structures in New Zealand. This includes developing and testing wall models with a different facing type (segmental or wrap-around facing), load cell instrumentation of all reinforcement layers, dynamic loading on the wall panel and the use of local soils as the backfill material. Lastly, the limitations of the experimental procedure and wall models should be understood; in particular, the similitude issues with regard to the soil stress and corresponding soil-geogrid interaction.
AN INVESTIGATION INTO THE SEISMIC PERFORMANCE AND PROGRESSIVE FAILURE MECHANISM OF MODEL GEOSYNTHETIC REINFORCED SOIL WALLS

A thesis submitted in partial fulfilment of the requirements for the Degree of Master of Engineering at the University of Canterbury

By Kelvin Loh

August 2013
ABSTRACT

Geosynthetic reinforced soil (GRS) walls involve the use of geosynthetic reinforcement (polymer material) within the retained backfill, forming a reinforced soil block where transmission of overturning and sliding forces on the wall to the backfill occurs. Key advantages of GRS systems include the reduced need for large foundations, cost reduction (up to 50%), lower environmental costs, faster construction and significantly improved seismic performance as observed in previous earthquakes. Design methods in New Zealand have not been well established and as a result, GRS structures do not have a uniform level of seismic and static resistance; hence involve different risks of failure. Further research is required to better understand the seismic behaviour of GRS structures to advance design practices.

The experimental study of this research involved a series of twelve 1-g shake table tests on reduced-scale (1:5) GRS wall models using the University of Canterbury shake-table. The seismic excitation of the models was unidirectional sinusoidal input motion with a predominant frequency of 5Hz and 10s duration. Seismic excitation of the model commenced at an acceleration amplitude level of 0.1g and was incrementally increased by 0.1g in subsequent excitation levels up to failure (excessive displacement of the wall panel). The wall models were 900mm high with a full-height rigid facing panel and five layers of Microgrid reinforcement (reinforcement spacing of 150mm). The wall panel toe was founded on a rigid foundation and was free to slide. The backfill deposit was constructed from dry Albany sand to a backfill relative density, \( Dr = 85\% \) or 50\% through model vibration.

The influence of GRS wall parameters such as reinforcement length and layout, backfill density and application of a 3kPa surcharge on the backfill surface was investigated in the testing sequence. Through extensive instrumentation of the wall models, the wall facing displacements, backfill accelerations, earth pressures and reinforcement loads were recorded at the varying levels of model excitation. Additionally, backfill deformation was also measured through high-speed imaging and Geotechnical Particle Image Velocimetry (GeoPIV) analysis. The GeoPIV analysis enabled the identification of the evolution of shear strains and volumetric strains within the backfill at low strain levels before failure of the wall thus allowing interpretations to be made regarding the strain development and shear band progression within the retained backfill.

Rotation about the wall toe was the predominant failure mechanism in all excitation level with sliding only significant in the last two excitation levels, resulting in a bi-linear displacement acceleration curve. An increase in acceleration amplification with increasing excitation was observed with amplification factors of up to 1.5 recorded. Maximum seismic and static horizontal earth pressures were recorded at failure and were recorded at the wall toe. The highest reinforcement load was recorded at the lowest (deepest in the backfill) reinforcement layer with a decrease in peak load observed at failure, possibly due to pullout failure of the reinforcement layer. Conversely, peak reinforcement load was recorded at failure for the top reinforcement layer.

The staggered reinforcement models exhibited greater wall stability than the uniform reinforcement models of L/H=0.75. However, similar critical accelerations were determined for the two wall models due to the coarseness of excitation level increments of 0.1g. The extended top reinforcements were found to restrict the rotational component of displacement and prevented the development of a
A preliminary shear band at the middle reinforcement layer, contributing positively to wall stability. Lower acceleration amplification factors were determined for the longer uniform reinforcement length models due to reduced model deformation. A greater distribution of reinforcement load towards the top two extended reinforcement layers was also observed in the staggered wall models.

An increase in model backfill density was observed to result in greater wall stability than an increase in uniform reinforcement length. Greater acceleration amplification was observed in looser backfill models due to their lower model stiffness. Due to greater confinement of the reinforcement layers, greater reinforcement loads were developed in higher density wall models with less wall movement required to engage the reinforcement layers and mobilise their resistance.

The application of surcharge on the backfill was observed to initially increase the wall stability due to greater normal stresses within the backfill but at greater excitation levels, the surcharge contribution to wall destabilising inertial forces outweighs its contribution to wall stability. As a result, no clear influence of surcharge on the critical acceleration of the wall models was observed. Lower acceleration amplification factors were observed for the surcharged models as the surcharge acts as a damper during excitation. The application of the surcharge also increases the magnitude of reinforcement load developed due to greater confinement and increased wall destabilising forces.

The rotation of the wall panel resulted in the progressive development of shears surface with depth that extended from the backfill surface to the ends of the reinforcement (edge of the reinforced soil block). The resultant failure plane would have extended from the backfill surface to the lowest reinforcement layer before developing at the toe of the wall, forming a two-wedge failure mechanism. This is confirmed by development of failure planes at the lowest reinforcement layer (deepest with the backfill) and at the wall toe observed at the critical acceleration level. Key observations of the effect of different wall parameters from the GeoPIV results are found to be in good agreement with conclusions developed from the other forms of instrumentation.

Further research is required to achieve the goal of developing seismic guidelines for GRS walls in geotechnical structures in New Zealand. This includes developing and testing wall models with a different facing type (segmental or wrap-around facing), load cell instrumentation of all reinforcement layers, dynamic loading on the wall panel and the use of local soils as the backfill material. Lastly, the limitations of the experimental procedure and wall models should be understood.
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Chapter 1.0 INTRODUCTION

1.1 Overview
In this thesis, results from an experimental study on the seismic performance of full-height rigid geosynthetic reinforced soil (GRS) or mechanically stabilised earth (MSE) walls are presented. The study consisted of 12 1-g reduced-scaled GRS wall model tests that were conducted at the University of Canterbury. Excitation of the wall models were achieved through a uni-directional shake table in the university’s structures laboratory. Key parameters such as the reinforcement length, reinforcement layout and the density of the retained backfill were systematically varied to study their influence on the seismic behaviour of the wall models. Responses of the wall model in the form of facing displacements, earth pressures, reinforcement loads, accelerations within the backfill and backfill deformation were measured during each test.

In particular, local deformations within the backfill were of interest in this study. Using a combination of high-speed image capture and analysis using the Geotechnical Particle Image Velocimetry (GeoPIV) software developed by White and Take (2002), a deeper understanding of the strain field development within the backfill can be achieved. A brief introduction of the GeoPIV software is presented in Chapter 5.

In this chapter, an outline of the history and development of the use of geosynthetics in retaining walls is first presented. Additionally, the typical characteristics and types of GRS walls as well as preliminary design considerations for GRS walls are introduced. This is followed by a review of the performance of GRS walls in recent earthquakes and their advantages compared to conventional retaining walls in Section 1.2. Objectives and organisational structure of this study are presented in Section 1.3.

1.2 Geosynthetic-reinforced soil walls

1.2.1 History of GRS walls
Although the use of geosynthetics has only become prominent in the past three decades, the concept of reinforced fill is not new. Inclusions to improve soil have been used since prehistoric times. Examples of this are the Agar-Quf Ziggurat, where woven mats of reeds were used in between the clay bricks and the Great Wall of China, where clay and gravel mixtures with tamarisk branches were used (GEO, 2002). Early examples of man-made soil reinforcement are earth and branches dikes that have been used in China for at least 1,000 years and along the Mississippi River in the 1880s (FHWA, 2001).

The modern form of soil reinforcement for retaining wall construction was pioneered by Henri Vidal, the French architect and engineer in the early 1960s. Vidal’s concept was for a composite material formed from flat reinforcements laid horizontally in a frictional fill with the interaction between the fill and reinforcement solely friction generated by gravity (G.E.O, 2002). His research lead to the development of Reinforced Earth, a system in which steel reinforcement strips were used (FHWA, 2001).
The use of geosynthetic reinforcement only became prominent in the early 1970s. The first geotextile reinforced wall was constructed in France, 1971 with the first of this type constructed in USA in 1974 (FHWA, 2001). In Japan, the first project using GRS was commenced in 1988 and use of GRS on a wider scale only started in 1992 (Murashev, 2003). According to the FHWA (2001), the first use of geogrids in soil reinforcement was in 1981.

As a result, design methods for GRS walls have not been well established. Currently, New Zealand geotechnical engineers use several different overseas standards and design guidelines for manufacturers to design GRS structures (Murashev, 2003). This results in GRS structures in New Zealand having different levels of seismic and static resistance thus, different risks of failure. Furthermore, there does not seem to be a widely held consensus for seismic design procedures for GRS walls.

### 1.2.2 Characteristics of GRS walls

Figure 1-1 shows key features that differentiate GRS walls from conventional cantilever retaining walls as well as some applications of GRS walls (FHWA, 2001). Key features of GRS walls include: geosynthetic reinforcement layers within the reinforced soil block; wall facing type; and geosynthetic reinforcement connections. Note that for GRS walls, reduction in the concrete and foundation size is achieved compared to the cantilever retaining wall. Key features of GRS walls are further elaborated in the following paragraphs.

![Figure 1-1: (a) Cross-section of typical cantilever retaining wall and GRS retaining wall; (b) Cross-section of typical cantilever wall and GRS wall application for bridge abutments (FHWA, 2001).](image)
Geosynthetic Reinforcement within reinforced soil block – Polymeric material in the form of geotextiles (flexible sheets) or geogrids (mesh-like pattern with intersecting elements). The geosynthetic reinforcement transmits overturning and sliding forces acting on the wall to the backfill through its tensile capacity and frictional resistance. Engineering backfill is typically used for the reinforced soil block with a relative density of about 90%.

Wall facing type – The wall facing types most commonly used in NZ are the wrap-around facing and the segmental block facing type. Further details are provided in Section 1.2.3.

Reinforcement connections – Jointing of geosynthetics is required where the geosynthetic widths or lengths required are greater than that supplied in one roll. Jointing can be achieved mechanically (stapling or sewing), chemically (adhesive bond) or by using a bodkin joint or sewn joint for geogrids (bar or cord through apertures) (Shukla, 2002). Connections between the geosynthetic and the wall facing can be rigid, as a mechanical connection to concrete panel (FHR walls) or non-rigid, typically frictional when the reinforcement is placed between segmental blocks in a segmental retaining wall.

1.2.3 Type of GRS walls

The terminology used to classify the different types of GRS walls has not been well regulated and this has led to the use of different classifications by different guidelines as shown in Table 1-1 below. The design codes referenced are: Federal Highway Administration (FHWA) guidelines in USA (2001) and the NZ guidelines by Murashev (2003).

<table>
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<th>Type of GRS wall</th>
<th>Description</th>
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<td>Segmental Precast Concrete Panel (FHWA, NZ guidelines)</td>
<td>Discrete Concrete Panels connected with shear pins</td>
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<tr>
<td>Dry Cast Modular Block walls (FHWA)/Proprietary segmental precast concrete units (NZ guidelines)</td>
<td>Relatively small concrete units typically Dry-stacked with vertical adjacent units connected with shear pins, lips or keys</td>
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<tr>
<td>Full-height Rigid system (NZ guidelines)¹</td>
<td>Precast Concrete Panels typically erected and propped before backfilling</td>
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<td>Staged-construction Full-height Rigid (FHR) facing (Tatsuoka, 2008)</td>
<td>Thin and lightly steel-reinforced concrete facing constructed by cast-in-place concrete directly on a wrap-around wall system</td>
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<tr>
<td>Wrap-around system (NZ guidelines) /Geosynthetic Facing (FHWA)</td>
<td>Geotextile reinforcement looped around facing to form exposed face of retaining wall</td>
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¹ FHR systems were used in all of the wall models tested in this thesis

Of the systems described in the Table 1-1 above, the systems of note are the Full-height Rigid (FHR) and the Staged-construction FHR facing types, the former being the focus of this study. The staged-construction FHR facing type walls has become standard retaining wall construction technology for railways including bullet train lines in Japan. This is due to its cost-effectiveness as expensive temporary structures are not needed and its high strength against concentrated loads applied to the top of the facing as it is continuous in both the vertical and lateral direction (Tatsuoka, 2008).
An important feature of wrap-around systems is that the geosynthetic is subject to UV light degradation due to the exposed face of the wall. Also, Ling et al. (2001) conducting a post-earthquake review of GRS structures after the Chi-Chi earthquake of 1999 found that a number of segmental block retaining walls had failed due to deformation of the modular block facing. A 1998 survey of all GRS structures constructed in New Zealand and found that the predominant facing was wrap-around facing and 30% of walls were segmental precast concrete blocks (Murashev, 1998). A more recent survey of types of GRS walls in New Zealand has not been conducted.

1.2.4 Key Advantages of GRS walls
Previous studies on the performance of GRS walls in recent seismic events have found that significant damage to conventional retaining wall structures occurred whilst reinforced soil structures demonstrated limited to no damage (Jackson, 2010). A summary of the performances of GRS walls in earthquakes in Northridge (1994), Kobe (1995) and Chi-Chi (1999) can be found in Jackson (2010).

Discounting the improved performance of GRS walls under seismic loading, several other key advantages of GRS walls compared to conventional retaining walls are presented as follows:

- A reduced need for large foundations as the reinforcement layers act to support the GRS wall; reducing the overturning moments and sliding moments acting on the wall toe.
- Studies by Koerner et al. (1998) and Jones (1996) showed that construction of GRS walls led to a cost reduction of 50% in the USA and 75% in the UK compared to conventional cantilever retaining walls as reported by Jackson (2010).
- Jones (1996) also found that based on ecological parameters such as energy, labour, emissions and despoiling of the land, the environmental cost of GRS walls is approximately 30% cheaper compared to conventional cantilever retaining walls.
- Tatsuoka (2008) also reports that construction of GRS walls with a FHR facing not only greatly lowered construction cost and offered higher stability but also much faster construction and a significant reduction of earthwork was achieved compared to conventional gravity type retaining walls.
- Another key advantage of GRS walls with a FHR facing is that the FHR facing is able to function as a foundation for the super-structure (as a bridge abutment). In Tatsuoka (2008), the construction of a GRS wall with a FHR facing on a Loam soil deposit for a railway in Tokyo (without using a pile-foundation) is presented with no reported significant settlement of the bridge girder due to the train load.

Further details of the advantages of FHR facing GRS retaining walls and their possible applications for bridge abutments can be found in Tatsuoka (2008).

1.2.5 Design of GRS walls
The performance of GRS structures is typically considered in terms of Ultimate Limit State and Serviceability Limit State.

1.2.5.1 Serviceability Limit State
Due to the complex stress-strain state of the reinforced soil mass, currently no commonly accepted methods to design GRS structures for the serviceability limit state (SLS) are available (Murashev, 2003). The NZ guidelines recommend conventional settlement analyses to be undertaken to ensure that the settlement of the structure during and after construction is less than the allowable
settlement for the associated structures. The lateral displacements of GRS structures should also be assessed.

Internal strain criteria may be used to assess the performance of GRS structures but these requirements are not included in most of the existing design guidelines. According to the NZ guidelines, adequate Ultimate Limit State design and construction procedures typically result in satisfactory post-construction behaviour in terms of internal strains.

1.2.5.2 Ultimate Limit State

The common practice in designing GRS structures for the ultimate limit state (ULS) is to use limit equilibrium analysis which checks the overall stability of the structure. The types of stability analysis consist of external, internal and combined stability:

i. **External Stability**: Overall Stability of the reinforced soil mass considered as a whole; involves failure planes entirely outside or at the boundary of the reinforced block

ii. **Internal Stability**: Involves failure planes located entirely within the reinforced block.

iii. **Combined Stability**: Critical slip surface partially outside and partially inside the reinforced block. (Not taken into account in the NZ guidelines).

According to Murashev (1998), modes of failure associated with external stability analyses are shown in Figure 1-2 and include: (a) sliding; (b) overturning; (c) bearing capacity failure and (d) deep-seated failure. Similarly, the failure modes affiliated with internal stability of the GRS wall are presented in Figure 1-3 and include: (a) Pullout failure of the reinforcement layer; (b) Rupture of the reinforcement layer and (c) Internal sliding along a reinforcement layer (Murashev, 2003).

![Figure 1-2: Failure modes arising from external instability (Murashev, 1998).](image-url)
1.3 Details of this study

1.3.1 Research Objectives

Although good seismic performance of GRS structures has been observed, this is likely due to their underlying stability under high seismic loads and conservatism in static design. Therefore, further research is required to better understand the seismic behaviour of GRS structures and is the main focus of this study. The influence of key parameters such as reinforcement length, reinforcement layout, backfill density and surcharging is investigated. The specific objectives of the research are as follows:

- Refine and incorporate additional instrumentation to GRS wall shake-table test procedures developed by Jackson (2010)
- Determine the influence of reinforcement length and layout, backfill density and surcharging on the seismic behaviour of GRS walls through evaluation of earth pressures, reinforcement load development, backfill accelerations and wall facing displacements.
- Identify failure mechanisms and pre-failure deformational characteristics of the backfill by utilizing GeoPIV to examine strain development at specific regions of the retained backfill during dynamic excitation of the model.
- Identify key issues for further research studies.

To accomplish these objectives, a series of 1-g shake table tests of twelve GRS wall models was conducted at the University of Canterbury. These wall models were extensively instrumented with earth pressure cells, reinforcement load cells, accelerometers and potentiometers. The experimental results were then systematically analysed providing key findings on the deformational behaviour of GRS walls under dynamic excitation.
1.3.2 Organisation of Thesis

This thesis consists of 6 main chapters with Chapter 1 being the introductory chapter for the thesis.

In Chapter 2, a review of the current seismic design methodology for geosynthetic-reinforced soil walls is presented along with the effect of dynamic loading on the material properties within the GRS wall. Seismic design considerations such as acceleration amplification within the GRS backfill and critical acceleration of the GRS wall are also briefly introduced. Additionally, a main portion of the chapter discusses findings of previously conducted studies regarding the effect of key wall parameters on the seismic behaviour of GRS walls. These key parameters include the geogrid reinforcement length and layout, backfill density, wall inclination and wall facing type. The failure mechanisms identified for GRS walls are also discussed. Lastly, Chapter 2 also briefly discusses the accuracy of current design guidelines.

Chapter 3 presents the experimental methodology employed in this study. The GRS wall model design and detailing such as model dimensions, backfill soil properties and geogrid reinforcement details are first presented. The shake-table used for model excitation and its motion dynamics are detailed in this chapter. Instrumentation type, layout and installation details to monitor key GRS wall responses such as wall facing displacement, reinforcement loads and accelerations are then discussed. The use of high-speed cameras to capture images of the backfill deformation for GeoPIV analysis is also touched on in this section. A breakdown of the GRS wall model construction is then presented with details regarding the sand deposit construction, initial bracing of the FHR wall panel and so on. At the end of the chapter, a tabulated summary of the global backfill densities achieved for the twelve wall models (KL1 to KL12) constructed is presented.

The experimental results from the 1-g shake table tests are presented in Chapter 4. Firstly, the reliability of experimental results is discussed along with any potential issues affecting the results. This is followed with a presentation of experimental results such as wall displacements, reinforcement loads and acceleration amplification of a representative wall model chosen. Through this, the typical behaviour of a GRS wall model observed in the experiments is presented. Lastly, the influence of GRS wall parameters on the seismic performance of the GRS wall models is examined through a comparison of the experimental results.

In Chapter 5, the results from the GeoPIV analysis that was used to determine and quantify the backfill deformation are presented. A brief introduction to the GeoPIV methodology is presented along with discussion regarding the application of GeoPIV in this research. Similarly to Chapter 4, the backfill strain development for a representative GRS wall model is first presented followed by discussion on the effect of different GRS wall parameters on the backfill strain development.

Conclusions and recommendations arising from this research study are presented in Chapter 6.
1.4 References


Chapter 2.0 LITERATURE REVIEW

2.1 Introduction
In this chapter, we will mainly focus on the seismic aspects of geosynthetic-reinforced soil walls. A review of the current static design methodology may be found in the New Zealand Guidelines (Murashev, 2003). In Section 2.2, properties of GRS wall components are discussed along with a review of the current seismic design methodologies for GRS walls. In addition to this, response characteristics of interest during seismic loading of a GRS wall are also discussed such as acceleration amplification and critical acceleration.

Section 2.3 presents a review of results and findings of previous GRS wall studies with regard to the influence of key wall parameters on the seismic response of GRS walls such as wall lateral deformation, reinforcement loads, acceleration amplification and so on. Key wall parameters discussed include the reinforcement length, backfill density and wall facing properties. Lastly, the accuracy of current design guidelines in predicting the behaviour of GRS walls is briefly discussed in Section 2.4.

2.2 Seismic Design Aspects of GRS Walls

2.2.1 Material properties
Under dynamic loading, properties of components within GRS wall structures maybe influenced by the rate of loading and have different cyclic loading responses. In this section, seismic design considerations for key GRS wall parameters such as the backfill soil, the geosynthetic reinforcement and the soil-geosynthetic interface and its influencing factors are discussed through a review of previous studies.

2.2.1.1 Soil
A review of the literature suggests that for dry cohesionless soils, the rate of loading (dynamic or static) has negligible effect on shear strength. Schimming and Saxe (1964) used a direct shear device to test Ottawa sand under both static and dynamic conditions and found no significant difference in shear strength envelopes recorded.

Conventional practice in using the displacement method or Newmark method is to assume that the cohesionless soil friction angle does not change during an earthquake. For pseudo-static methods, a value of \( \delta = 2\phi/3 \) for internal stability analyses and \( \delta = \phi \) for external stability analyses is assumed to be applicable for GRS walls (Bathurst and Cai, 1995). The choice of peak friction angle to be used in seismic analysis is consistent with the NCMA, AASHTO and FHWA guidelines for static design of GRS structures as under rapid loading, the strength of compacted, unsaturated cohesionless backfills are expected to be at least as great as the static value (Bathurst and Cai, 1995).

In the design, a value of \( \phi = 35^\circ \) is recommended for cohesionless sands and gravels (Tatsuoka et al., 1998). The authors note that the value chosen is likely to be closer to the residual strength of the material but argue that the selection of this value accounts for possible progressive failure of the backfill soil and uncertainties such as soil compaction levels. The AASHTO (1998) guidelines recommend that the friction angle within the reinforced backfill does not exceed \( \phi = 34^\circ \). Allen
and Bathurst (2002) in their study of internal stability of GRS walls concluded that the low reinforcement strains and loads measured to date in geosynthetic walls point to the desirability of using peak soil shear strengths rather than constant volume shear strengths for design purposes. This is agreed with by other studies (Zornberg, 2002).

### 2.2.1.2 Geosynthetic Reinforcement

In-isolation monotonic load tests (in-air load tests instead of in-soil load tests where the geogrid is able to interact with the soil) were carried out on typical polymeric geogrid reinforcement materials (Bathurst and Cai, 1994) and showed that high density polyethylene, HDPE geogrids were more sensitive to the rate of loading than woven polyester, PET geogrids. The available strength and stiffness of geosynthetic reinforcement products under seismic loading were not less than conventional estimates in static load environments and could be even much greater. As observed in Figure 2-1, where $T$ is the tensile load in (kN/m) and $\epsilon$ is the strain experienced by the geogrid along its length, the cyclic load strain behaviour of typical geogrid is defined by: (1) non-linear hysteresis unload-reload loops; (2) a load-strain cap that is tangent to all initial unload-reload hysteresis curves. These unload-reload curves have a different stiffness than the initial stiffness of the geogrid.

Moraci and Montanelli (1997) carried out cyclic tension tests on HDPE geogrids at frequencies in the range of 0.1-1.0Hz and at different load amplitudes. Their results showed that the unload-reload stiffness value decreased with increasing load amplitude and increased with greater loading frequency. They also observed that unload-reload stiffness of the HDPE geogrids for load amplitudes less than 60% of the reference tensile strength was approximately 1.5 to 2 times the secant stiffness from monotonic tensile strength tests (Moraci and Montanelli, 1997). Similar findings are also reported by Ling et al. (1998) who found that the reload stiffness of all polymeric materials (HDPE; PET; polypropylene, PP) examined at any given load level increased with the number of loading cycles. In terms of post-cyclic tensile strength, the tensile strength of HDPE and PET geogrids increased with the number of cyclic loads and load amplitude, in contrast to the PP geogrid which showed no significant change in the tensile strength during cyclic loading (Ling et al., 1998).

![Figure 2-1: Characteristics of cyclic response of the HDPE geogrid specimen under full cycling loading (Bathurst and Cai, 1994).](image-url)
Furthermore, some monotonic in-soil tensile tests have indicated that increasing confining pressures increase the modulus of needle-punched non-woven geotextiles and may increase ultimate strength as well (Shukla, 2002). McGown et al. (1995) performed a series of low frequency, in-soil cyclic tests similar to Bathurst and Cai (1994) and reported that the stresses and permanent strains may be ‘locked-in’ in the reinforcement due to repeated tensile loading, resulting in a stiffer reinforcement response than that for an in-air test.

An implication of these findings for GRS design is that we might be greatly underestimating the reinforcement stiffness and tensile working strength of these materials within the GRS structure and under dynamic loading which leads to an overly conservative design. In-air monotonic tensile tests of the materials may be taken as a lower bound on the reinforcement stiffness and tensile strength for HDPE and PET geogrids.

2.2.1.3 Soil-Geosynthetic Interface

From GRS structures, the soil-geosynthetic interaction is important as it must be sufficient to prevent the soil from sliding over the geosynthetic surface or the pullout of geosynthetic reinforcement when tensile load is mobilised in the reinforcement. We will mainly focus on geogrids due to its major use in GRS structures.

Palmeria & Milligan (1989) attempted to define the mechanisms of interaction of the soil and the geogrid in a GRS structure. The interaction mechanisms deduced are shown in Figure 2-2. Model studies of GRS walls have shown that the failure plane of the wall does not form across the geogrid reinforcement but form along the reinforcement and along the interface between the reinforced backfill and unreinforced backfill (Jackson, 2010). Therefore, the interaction mechanism where the reinforcement is sheared out-of-plane is not applicable for GRS walls (shown as Mechanism (C) in Figure 2-2). The two interaction mechanisms of interest in this report are the direct sliding resistance (Mechanism A) and the bond strength (or pullout strength) of the geogrid reinforcement (a combination of Mechanism B & D).

![Figure 2-2: Interaction Mechanisms in a GRS wall](Palmeira, 2009)modified from (Palmeira and Milligan, 1989).
Palmeria (2009) recommended specific tests for the different mechanisms formed. However, all the test types have limitations in simulating actual conditions in a GRS structure. The in-plane direct shear test (representing direct sliding resistance, A) and the pull out test (representing bond strength, B & D) are focused on in this report as they are both widely accepted to be the closest tests to modelling soil-geosynthetic interaction in GRS walls. Other tests include the direct shear test with inclined geogrid (for Mechanism C), the in-soil tensile test (for Mechanism B only) and the ramp test (for Mechanism A).

The main mechanisms of the soil-geogrid interaction comprises of: (i) frictional resistance between the soil and the plane surface of the geogrid; (ii) internal shear resistance provided by the interlock of soil particles in the openings of the geogrid; and (iii) the bearing resistance of the soil particles on the transverse ribs of the geogrid. Note that for geotextiles, only mechanism (i) provides the interface friction.

Frictional resistance between the soil and plane surface of the geogrid is measured conventionally for soils and construction materials. Potyondy (1961) carried out a comprehensive series of tests where he used a direct shear box test with a block of construction material in the lower half of the box and soil in the upper half. This resistance to sliding was defined as the skin friction angle, \( \delta \) (Potyondy, 1961). This test procedure has now become the standard test method for measuring friction between soil and plane reinforcement surfaces.

The frictional resistance due to the soil-geogrid interface can be defined as:

\[
T = 2WL \sigma_n' \sin \phi
\]

Where:
- \( f \) is the soil-reinforcement interaction coefficient which has different definitions, dependent on the interaction mechanism;
- \( \phi \) is the peak soil friction angle which is a function of effective stress and soil density;
- \( W \) & \( L \) are the width and length of the reinforcement respectively;
- \( \sigma_n' \) is the effective normal stress at the interface.

There are relatively few findings in the literature regarding dynamic testing of the soil-geogrid interface. In terms of the direct sliding resistance, O’Rourke et al. (1990) tested a single specimen of HDPE sheet in combination with Ottawa sand at low confining pressure and showed that there was no reduction in interface shear strength with the number of shear applications. Based on this, it is reasonable to use results of monotonic loading direct shear tests for limit-equilibrium seismic design. This is confirmed by the findings of Fakharian and Evgin (1995) who conducted cyclic shear tests between sand and fine steel mesh surfaces and showed that monotonic and cyclic direct shear tests gave the same values of peak and residual interface shear strength.

For bond strength testing, typically tested by the pullout test, conflicting results were found in the literature. Raju (1995) and Yasuda et al. (1992) reported that the magnitude of peak cyclic load to cause pullout failure is greater than the load required for static pullout failure. This conflict with the findings in Min, et al. (1995) who used biaxial PP geogrid and reported that soil-geogrid interaction coefficient reduced by about 20% due to repeated loading compared to the interaction coefficient back-calculated from single load pullout tests. The conflict in results could be attributed to the
anchorage length used to back-calculate interaction coefficient values and further investigations will have to be carried out to determine the influence of dynamic loading on pullout tests.

AASHTO (1998) and FHWA (1996) recommend reduction in pullout interaction coefficient to 80% of that used in static design which is based on pullout tests on steel strip reinforcement. However, the AASHTO and FHWA guidelines already compensate for this by reducing the factors of safety against pullout failure in limit-equilibrium design to 75% of the static design values (Shukla, 2002). This is another indication of overly conservative design recommendations which could result in unnecessary costs in construction.

2.2.1.3.1 Direct Sliding Resistance

Direct sliding resistance for geogrids is best represented by a direct shear test with soil in the upper and lower containers of the shear box and geogrid fixed along the predefined shearing plane (Liu, Ho and Huang, 2009). For direct sliding resistance, the shear strength of the soil-geogrid interface is assumed to be due to only two mechanisms: Mechanism (i), the frictional resistance between the soil and plane surface of the geogrid and Mechanism (ii), the internal shear resistance from the interlock of soil particles within the openings in the geogrid as balancing of bearing stresses across the grid apertures occurs due to shearing in opposite directions (Jewell et al., 1985). The ratio of peak shear strengths of the soil-soil and soil-geogrid interfaces from direct shear box tests is typically used to determine the direct sliding coefficient, \( f_{ds} \). It is noted that there is some ambiguity of the shear box test results, largely due to the lack of knowledge of the principal stress directions in the shear box and the fact that the loading plate in the shear box tends to tilt during shear leading to a non-uniform vertical stress distribution (Ingold 1982).

From Jewell et al. (1985), the coefficient of direct sliding resistance, \( f_{ds} \) can be expressed as:

\[
\frac{f_{ds}}{1} = 1 - \alpha_{ds} \left( 1 - \frac{\tan \delta}{\tan \phi_{ds}} \right) = \frac{\tau_{soil/plane\ reinforcement}}{\tau_{soil}}
\]

Where:
- \( \alpha_{ds} \) represents the contact fraction of grid surface area and soil (depends on the relative size of soil particles to grid apertures);
- \( \delta \) is the skin friction angle for soil on plane reinforcement surfaces;
- \( \phi_{ds} \) is the soil friction angle in direct shear;
- \( \tau_{soil/plane\ reinforcement} \) is the peak shear strength obtained from direct shear tests of the soil-geogrid interface;
- \( \tau_{soil} \) is the internal peak shear strength of the soil.

For the design of GRS walls or slopes where the soil fill particles penetrate the grid apertures, Jewell et al. (1985) recommends that a suitable value of maximum direct sliding resistance could be found by adopting \( \alpha_{ds} = \alpha_{s} \), where \( \alpha_{s} \) is the fraction of solid surface area in a grid.

\[
\frac{f_{ds}}{1} = 1 - \alpha_{ds} \left( 1 - \frac{\tau_{soil/plane\ reinforcement}}{\tau_{soil}} \right)
\]

Where:
- \( \tau_{soil/plane\ reinforcement} \) is the peak shear strength obtained from direct shear tests of the soil-plane reinforcement interface (Potyondy 1961);
- \( \tau_{soil} \) is the internal peak shear strength of the soil.
Liu et al. (2009) used the above Eqn (2-3), modified from Eqn (2-2), to determine the direct sliding resistance coefficient for geogrids. The equation uses the peak shear strength values of the soil against plane reinforcement (determined in a similar method with Potyondy (1961)) to determine the direct sliding resistance coefficient for the geogrid. Using this model, typical direct shear tests between the soil and geogrid is not required.

This theoretical model of $f_{ds}$ was validated by comparing its results with test results presented by Jewell et al. (1985) and test results of $f_{ds}$ from the researchers experiments. A similar general trend of the test results were achieved when plotted against geogrid aperture width/D$_{50}$ and for different soil/PET geogrids (Liu et al., 2009). However, the model overestimated the $f_{ds}$ values for the interfaces with geogrid aperture width/D$_{50}$ ranging from between 1 to 100. This overestimation was attributed to the small value of $\alpha_{ds}$, the contact fraction of the grid surface area and the soil, as a small value of $\alpha_{ds}$ would result in a large theoretical value of $f_{ds}$.

Several studies have been conducted to determine the contribution of bearing stresses in direct sliding resistance. Jewell et al. (1985) claimed that the bearing stresses provided by the transverse ribs could be ignored because (1) the soil in the upper and lower halves of the grid apertures would shear in opposite directions so the bearing stresses would be balanced across a transverse rib and (2) for granular soils, the rupture surface is above the top of the grid thus the soil contained in the grid apertures would not displace relative to the transverse rib. This was disputed by other researchers who stated that the geogrid apertures might be able to provide bearing resistance under direct shear mode (Bergado et al., 1993). Liu et al. (2009) performed large-scale direct shear tests on a variety of geogrids and found that the discrepancy between the model-predicted coefficient of direct sliding resistance, $f_{ds}$ from Eqn (2-3) and the test results increases with an increase in the total area of bearing ribs. This indicates some bearing contribution to the direct sliding resistance. However, test results showed that the bearing contribution is at most about 10% of the total shear resistance in all tests conducted. Therefore, it would be not overly conservative to ignore the bearing contribution to the direct sliding resistance.

2.2.1.3.2 Bond Strength

The bond strength (or pullout strength) of the geogrid reinforcement is best determined by the pullout test. Since there is no relative movement of the soil on either sides of the geogrid in the pullout test, there is no shear resistance provided by the soil interlock between the apertures of the geogrid (Jewell et al. 1985). Therefore, only two of the three interaction mechanisms previously mentioned in Section 2.2.1.3 applies; Mechanism (i), frictional resistance between the soil and plane geogrid surface and Mechanism (iii), bearing resistance of the soil particles on transverse ribs of the geogrid. The contribution of bearing stresses is confirmed by pullout tests which showed that the transverse members of the geogrid were responsible for a significant fraction of the maximum pullout load of the geogrid (Teixeira, 2003). It is noted that there is a likely difficulty in interpretation of test data due to the extensibility of the reinforcement which results in non-uniform distribution of bond stresses along the sample (Ingold, 1982).

Bearing stresses acting on the geogrid can be expressed as a function of the normal effective stress and is typically determined from pullout tests. Two theoretical relationships for bearing stresses can be determined based on two different assumptions of the bearing capacity factor (Jewell et al. 1985).
1. Assuming a **punching shear failure mode** for the reinforcement bearing resistance, the normalized bearing stress is:

\[
\frac{\sigma'_b}{\sigma'_n} = N_q = e\left(\pi \tan \phi\right) \tan^2\left(\frac{\pi}{4} + \frac{\phi}{2}\right)
\]  

(2-4)

2. Assuming the **conventional stress characteristic field for a foundation** rotated to the horizontal, expressed as:

\[
\frac{\sigma'_b}{\sigma'_n} = N_q = e\left[\pi \tan \phi\right] \tan^2\left(\frac{\pi}{4} + \frac{\phi}{2}\right)
\]  

(2-5)

Where:

- \(\sigma'_b\) is the soil bearing stress ratio where:
  - \(\sigma'_b\) is the bearing stress & \(\sigma'_n\) is the effective normal stress applied;
- \(\phi\) is the soil friction angle in direct shear.

The two equations for bearing stress ratio produce a lower (Eqn 2-4) and upper bound (Eqn 2-5) for the bearing stress ratio respectively when plotted against the soil friction angle (Palmeira and Milligan, 1989; Jewell et al., 1985) with an observed change in failure mechanism from punching shear to generalized shear occurring when \(B/D_{50}\) exceeds 7.5 (Palmeira and Milligan, 1989). Most data collected from literature fall within the limits as shown in Figure 2-3.

The influence of member diameter to particle ratio, \(B/D_{50}\) on the bearing stress ratio normalized by the tangent of the soil friction angle, \(\sigma'_b/\sigma'_n \tan \phi\) is shown in Figure 2-4 (Palmeira and Milligan, 1989). Higher values of bearing stress would be likely if tests with low \(B/D_{50}\) were performed. The normalized bearing stress ratio appears to be constant for a \(B/D_{50}\) greater than 15, which agrees well with data presented by Kerisel (1972) on the effect of scale in bearing capacity problems. It is noted that rectangular grid bars have a higher bearing stress ratio compared to circular grid bars.

![Figure 2-3: Influence of soil friction angle on bearing stress ratio (Palmeira and Milligan, 1989).](image)

In all tests depth to member height ratio was greater than 5 (deep anchor).
An expression for the bond strength coefficient, $f_b$ (also represented as $F^*$ in the NZ guidelines) is presented by Jewell et al. (1985):

$$f_b = f_s + f_{bearing} = \frac{\tan \delta}{\tan \phi} + \frac{1}{2} \frac{\sigma_b'}{\sigma_n'} \frac{\alpha_b}{\tan \phi} \frac{B}{S}$$  \hspace{1cm} (2-6)

Where:
- $f_s$ represents the contribution from soil and geogrid in direct shear;
- $f_{bearing}$ represents the contribution from soil bearing;
- $\alpha_b$ is the fraction of grid width available for bearing;
- $B$ is the bearing member thickness;
- $S$ is the spacing between bearing members.

Another aspect of interaction between the soil and geogrid in pull-out type failures is the possibility of interference between the grid transverse members. Photo-elastic studies have shown clearly that load distribution between the grid transverse members is uniform only if the members are sufficiently far apart from each other (Dyer, 1985). Accounting for interference between bearing members, we can model bearing resistance as (Palmeira and Milligan, 1989):

$$f_{bearing} = (1 - DI) \frac{1}{2} \frac{\sigma_b'}{\sigma_n'} \frac{\alpha_b}{\tan \phi} \frac{B}{S}$$  \hspace{1cm} (2-7)

Where:
- $DI$ is the degree of interference which is a function of soil properties and the geometry of the bearing members.

Jewell (1990) replotted findings of Palmeria and Milligan (1989) by normalizing the bearing stress in a different way and expressed a relationship between the soil particle size and the bearing stress:

$$\frac{\sigma_b'}{\sigma_n'} = \left( \frac{\sigma_b'}{\sigma_n'} \right)_\infty \left( 2 - \frac{B}{10D_{50}} \right) \text{ when } \frac{B}{D_{50}} < 10$$  \hspace{1cm} (2-8)
\[
\frac{\sigma'_b}{\sigma'_n} = \left( \frac{\sigma'_b}{\sigma'_n} \right)_{\infty} \quad \text{when} \quad \frac{B}{D_{50}} \geq 10
\]

Where: \( \left( \frac{\sigma'_b}{\sigma'_n} \right)_{\infty} \) is the bearing stress that is mobilized where the soil particle size is unimportant (when \( B/D_{50} \) > 10 in Figure 2-5).

Later, Jewell (1996) suggested that the bond strength coefficient be rewritten as:

\[
f_b = \alpha_s \frac{\tan \delta}{\tan \phi} + \frac{1}{2} F_1 F_2 \left( \frac{\sigma'_b}{\sigma'_n} \right)_{\infty} \frac{\alpha_b}{\tan \phi} \frac{B}{S}
\]

Where:

- \( F_1 \) is the scale effect due to mean particle size, \( D_{50} \), where:
  \[
  F_1 = 2 - \frac{B}{10D_{50}} \quad \text{for} \quad \frac{B}{D_{50}} < 10
  \]
  \[
  F_1 = 1.0 \quad \text{for} \quad \frac{B}{D_{50}} > 10
  \]
- \( F_2 \) is the shape factor (\( F_2 = 1.0 \) for circular bars; \( F_2 = 1.2 \) for rectangular bars)

Figure 2-5: Influence of particle size, \( B/D_{50} \) on mobilized bearing stress (Jewell, 1996).
2.2.1.3.3 Influencing Factors of Soil-Geosynthetic Interaction

A. Soil Particle Size

The soil particle size distribution (PSD) is very important in the soil-geogrid interaction. Jewell et al. (1985) concluded that the direct sliding coefficient increases with soil particle size and has a maximum value when the grain size is similar to the geogrid apertures (Figure 2-6). This is in agreement with findings by Palmeria and Milligan (1989) discussed previously.

The qualitative effect of increasing soil particle size is explained in Jewell et al. (1985). For small particle sizes, the failure surface will form along the lateral surface of the transverse ribs, or for very fine particles, this failure surface will form along the lateral surface the longitudinal members as well. However, for a particle size similar to grid apertures, the soil particles are lodged against the transverse ribs and the failure surface is forced away from the grid and into the soil mass. At this stage, the soil-geogrid interaction coefficient will be at a maximum. Soil particle size beyond the aperture dimensions will result in a lower coefficient due to reduced penetration thus a reduced interlock of the particles through the grid apertures. Jewell et al. (1985) also recommended the ratio for geogrids used as soil reinforcement:

\[
\frac{\text{Minimum aperture dimension}}{\text{Average soil particle size, } D_{50}} \geq 3
\]

(2-10)

Figure 2-6: Influence of particle size on the direct sliding resistance (Jewell et al., 1985).
B. Confinement Stress

Confinement stress plays a major role in soil-geogrid interaction due to its influence on the shear strength of the soil. Furthermore, studies have demonstrated that enhanced pullout resistance is achieved due to development of a 3-D interaction mechanism at the edges of the geogrid shown in Figure 2-7 which is a result of the confinement of the dilating zone of soil around the reinforcement (Alfaro et al. 1995). Results showed a calculated localised increase in normal stress due to dilatancy of 85% and 33% at a normal stress of 20kN/m² and 30 kN/m², but this became negligible under an applied normal stress of 50 kN/m². These results show that an increase in the normal stresses at the edges of the geogrid due to the restrained dilatancy of the soil occurs and it is especially prominent at low confinement stresses for more dilative soils (i.e. dense soils).

Lopes (1998) also conducted pullout tests using the same sand density and geogrid reinforcement but at different confining stresses, 24.5kPa and 38kPa. The increase of confinement stress by 55% lead to an overall increase in shear strength mobilized at the soil-geogrid interface and an improvement in interface shear resistance of about 11% (Shukla, 2002).

![Figure 2-7: Conceptualized pullout interaction mechanism of strip reinforcement: shear stress and strain mobilised around (a) wide strip, and (b) narrow strip: distribution of normal stress on (c) wide strip and (d) narrow strip (Alfaro et al. 1995).](image-url)
C. Soil density
Soil density affects the soil-geogrid interaction in a similar way as confinement stress. As mentioned before, dense soils tend to dilate more than loose soils, reaching higher peak strengths due to greater interlock between the soil particles. This dilatant behaviour is confined by the reinforcement leading to an increase normal stress around the geogrid thus increasing the soil-geogrid interface strength (Alfaro et al., 1995). Lopes and Ladeira (1996) performed pullout tests of a geogrid in two sand samples of different density, 50% and 86%. For the looser sand sample, the geogrids failed by a lack of adherence of the sand and reinforcement. For the dense sample, the soil-reinforcement interface maintained its integrity and the reinforcement failed through a lack of tensile strength. Furthermore, strain gauges on the reinforcement indicated that for the higher sample density tested, only one third of the inclusion length contributed to shear resistance, and the adherence length decreased with soil density (Lopes and Ladeira, 1996).

D. Geosynthetic structure – Interference between bearing members
As discussed in Section 2.2.1.3.2, photoelastic studies by Dyer (1985) have shown that an individual transverse bearing rib alters the state of stress in the region behind it, thus affecting the stress distribution on the following ribs. Palmeria & Milligan (1989) quantified the corresponding loss in maximum pull-out load by the Degree of Interference (DI) by comparing the pull-out load for a given geogrid with the pullout load of an idealized geogrid where no interference occurs (Eqn 2-7). Tests performed by Palmeria & Milligan (1989) on circular and square transverse ribs showed that there is an improvement in bearing stress of 20% for square transverse ribs.

2.2.2 Seismic considerations for GRS walls
Two key parameters that arise from the seismic loading of GRS walls is the critical acceleration of the GRS wall and the acceleration amplification or attenuation observed within the retained backfill.

2.2.2.1 Critical Acceleration
Bracegirdle (1980) defined critical acceleration as “the horizontal pseudo-static acceleration acting uniformly over the structure to achieve limiting equilibrium”. At critical acceleration, the dynamic factor of safety against failure of the wall, typically sliding or rotation, will be just slightly less than 1 and permanent displacements will occur. Therefore, critical acceleration is an important measure of GRS wall stability; a high critical acceleration indicates higher stability of the wall during an earthquake and higher accelerations needed to trigger instability (movement of the wall).

Researchers tend to have different definitions of critical acceleration depending on their model set-up. However, typically, critical acceleration is defined as the point on the displacement-acceleration curve where accelerations beyond this point result in a sharp increase in lateral displacement (El-Emam & Bathurst, 2007; Jackson, 2010). The critical acceleration value is important in model wall studies as a benchmark to compare stability of different types of walls.

2.2.2.2 Amplification of motion
Previous studies on reduced-scale model tests have shown that the magnitude of ground motion changes within the structure. Fairless (1989) observed amplification factors of 3.0 at the top of 1/6 scale model walls. Telekes et al. (1994) also observed amplification factors of similar amplitude except when resonance was achieved and amplification factors greater than 3.0 were observed. Acceleration amplification is also reported in Bathurst and Hatami (1998) and El-Emam and Bathurst (2004, 2007).
The amplification for the single degree of freedom model as a function of damping ratio and frequency ratio is shown in Figure 2-8. From this figure, a resonance condition occurs when the applied frequency of base excitation is equal to the natural frequency of the single degree of freedom model ($\nu/w_0 = 1$) which leads to amplification of motion. During seismic excitation, strain development leads to a reduction in soil stiffness corresponding to a decrease in the natural frequency of the soil structure ($w_0 = \sqrt{k/m}$).

El-Emam and Bathurst (2007) reported that prior to the estimated critical acceleration value, the outward acceleration amplification of the model walls were relatively small (between 1.0 and 1.5) but increased significantly thereafter. This corresponds to an increased loss in soil stiffness beyond the critical acceleration due to permanent deformation of the structure.

The influence of wall parameters on the acceleration amplification is discussed further in Section 2.3.5. However, it is important to note that El-Emam and Bathurst (2007) compared the AASHTO/FHWA predicted acceleration amplification values with measured values and found that the trend predicted is opposite to the trend observed from experimental data. Amplification factors predicted using the design codes indicate that the amplification decreases with input base acceleration but the opposite was observed in the experiments. This result implies the inaccuracy of current design guidelines in accounting for acceleration amplifications within the GRS wall structure. However, note that these results arose from scaled-down model studies of GRS walls. Further full-scale experiments would have to be performed to obtain acceleration amplification results for prototype walls; thus confirming the accuracy of the design codes in predicting amplification factors.

### 2.2.3 Seismic Analysis and Design

Design of the GRS wall structures involves the assessment of the external stability relating to the identification of failure surfaces or sliding of the wall and the internal stability where the properties of the geosynthetic reinforcement are analysed. Note that the selection of a horizontal acceleration coefficient for design and site-specific peak ground acceleration is also important in seismic design and is not covered in this report.
2.2.3.1 External Stability

There are three approaches for evaluating the performance of retaining structures: (i) force-based pseudo-static methods; (ii) displacement-based sliding block methods and (iii) time-history finite element analysis.

The conventional pseudo-static approach is the Mononobe-Okabe method (M-O method) used by most design codes for seismic design of GRS walls (FHWA, 2001; Murashev, 2003). For sites having large backfill slope angles and high peak ground accelerations > 0.42g, the displacement method (commonly the Newmark method) is recommended, as solutions using the Mononobe-Okabe method are not possible or would lead to very conservative design (Murashev, 2003). Displacement-based methods such as the Newmark Method, enables the designer to predict the order-of-magnitude of displacement based on an estimation of critical acceleration.

The current AASHTO (2002) method restricts pseudo-static methods to peak horizontal ground accelerations < 0.3g instead of the 0.42g recommended in the NZ guidelines as previously mentioned. For ground motions greater than 0.3g, FHWA (2001) recommends that a reduced acceleration coefficient for $k_h$, determined from a Newmark analysis, is used in the pseudo-static analysis. However, the use of this coefficient is subjected to several conditions being met by the GRS wall which is further elaborated in the FHWA manual. This method results in a more economical structure by designing for a small tolerable displacement rather than no displacement which is assumed by the Mononobe-Okabe method.

2.2.3.1.1 Pseudo-static methods

The basic principle for pseudo-static methods is the assumption that the soil structure behaves as a rigid plastic material and is in a state of limit equilibrium under the action of the acceleration-induced inertia forces superimposed on the existing static forces. The pseudo-static analysis requires the input of a peak design acceleration to determine the seismic acceleration coefficient, $k_h$, geometry of the soil structure and Mohr-Coulomb strength parameters ($c'$ and $\phi$). Stability of the soil structure is then determined by the ratio of total resisting force to total destabilizing forces.

The M-O theory is a direct extension of the static Coulomb theory for pseudo-static conditions. In a M-O analysis, pseudo-static accelerations are applied to a Coulomb active wedge and the pseudo-static soil thrust is obtained from the force equilibrium of the wedge. The M-O theory is subjected to limitations of the pseudo-static analyses as well as the limitations of the Coulomb theory. The analysis ignores the deformation of the rigid block and dynamic changes in the peak ground acceleration during wave propagation. Furthermore, the analysis is also not suitable for soils that experience significant loss of strength (ie. liquefiable soils) (Kramer, 1996).

Driving forces acting on the GRS wall are due to the dynamic active soil force and the inertial forces from the backfill due to horizontal accelerations. Firstly, we define the driving forces acting on the wall.

The dynamic active soil force, $F_{AE}$:

$$F_{AE} = 0.5(1 \pm k_v)k_{AE}y_r h^2$$

(2-11)
Where: \( h \) is the height over which the force acts;
\( \gamma_r \) is the unit weight of the reinforced backfill;
\( K_{AE} \) is the Mononobe-Okabe earth pressure coefficient (Eqn (2-12));
\( k_v \) is a vertical seismic coefficient.

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

Where: \( \phi \) is the soil angle of friction;
\( \xi \) is the seismic inertia angle (= \( \tan^{-1} \frac{k_h}{1 \pm k_v} \));
\( \theta \) is the inclination of the wall face from horizontal;
\( \beta \) is the backfill slope angle (for broken backfill slope see Section 1.4.2.1);
\( k_v \) is the horizontal seismic coefficient (as a fraction of acceleration due to gravity).

The total dynamic active soil force can be broken down to two components, the static active soil force, \( F_A \) and the incremental seismic soil force, \( \Delta F_{AE} \) as shown in Figure 2-9.

\[
F_{AE} = F_A + \Delta F_{AE}
\]  

Where: \( F_A \) is the force due to active soil thrust;
\( \Delta F_{AE} \) is the increment of the seismic force (= \( 0.5 \Delta K_{AE} \gamma h^2 \));
\( \Delta K_{AE} \) = \((1 - k_v)K_{AE} - K_A\);
\( K_a \) = \( K_{af} \) for retained backfill and
\( = K_{ar} \) for reinforced backfill.

Figure 2-9: Inertia forces and dynamic pressure increments additional to static forces acting on a GRS wall during an earthquake (Murashev, 2003).
Additionally, if a soil surcharge is present above the reinforced backfill, the soil inertial forces due to seismic loading can be broken down to two components:

\[ F_{ir} = F_{ir} + F_{is} \]

Where:
- \( F_{ir} \) is the inertial force for the reinforced soil block \( (= 0.5a_{h,ext} \gamma_f H^2) \);
- \( F_{is} \) is the inertial force for soil surcharge above the reinforced backfill \( (= 0.125a_{h,ext} \gamma_f H^2 \tan \beta) \);
- \( a_{h,ext} \) is the horizontal seismic coefficient;
- \( \gamma_f \) is the unit weight of the backfill (kN/m³);
- \( H \) is the retained backfill height;
- \( \beta \) is the backfill slope angle.

Only the portion of the reinforced block that extends 0.5H beyond the wall face is used to calculate inertia forces based on the assumption that horizontal inertial forces induced will not reach peak values simultaneously during an earthquake. However, the inertia force due to the weight of the facing would have to be considered if the unit weight of the facing is substantially greater than the unit weight of the soil.

For the analyses for external stability and internal sliding stability, the dynamic force increment \( \Delta F_{AE} \) is recommended to be reduced by 50% (Murashev, 2003; NCMA, 1998). Reasons for this reduction are not stated in the NCMA manual or in the NZ guidelines but this is likely due to the conservatism of the adopted earth pressure distribution for the upper wall sections (as observed in El-Emam and Bathurst (2007)).

In another approach, Steedman and Zeng (1990) proposed using a simple pseudo-dynamic analysis of seismic earth pressures to account for phase difference and amplification effects within the retained backfill; reported as the Steedman-Zeng method in Kramer (1996).

### 2.2.3.1.2 Allowable Displacement Methods:

Although pseudo-static methods provide information about the forces acting on the GRS wall, the permanent deformation of the GRS wall is not considered. The displacement-based approach offers the advantage of a quantitative assessment of transient and permanent displacements under seismic loading.

Most displacement methods have been formulated based on the sliding block theory proposed by Newmark (Newmark, 1965). The basic principle of the Newmark Sliding Block analysis is that permanent displacement occurs whenever the seismic force (plus the static force) acting on the rigid soil mass exceeds the available resistance along the potential sliding surface. This point of exceedance defines the critical acceleration and is typically computed using a pseudo-static method of analysis. The accumulated permanent displacement is determined by double-integration of the acceleration time history with the critical acceleration as the reference datum. This is illustrated in Figure 2-10.
Cai and Bathurst (1996) reformulated a number of displacement methods and classified the methods based on two main categories depending on the characteristic seismic parameters referenced in each method. The authors use correlations between the dimensionless displacement term, \( \frac{d}{\left( \frac{v_m}{k_m g} \right)} \) and the critical acceleration ratio to compare the different methods for the estimate of permanent seismic displacement of soil structures. The first category of methods uses the peak ground acceleration \( (k_m g) \) and peak ground velocity \( (v_m) \) as characteristic parameters, (e.g. Newmark’s method, Franklin and Chang’s upper bound method and Richard and Elms upper bound method) and the second category of methods uses the peak ground acceleration \( (k_m g) \) and predominant period \( (T) \) of the ground acceleration spectrum (e.g. Sarma’s method, Makdisi and Seed’s method, Yegian et al.’s method and Ambraseys and Menu’s method) (Cai and Bathurst, 1996). Ambraseys and Menu’s method and Yegian et al.’s method are considered to give better estimates of permanent displacement of the structures as the methods provide the probability of exceedance for an expected permanent displacement.

The first category of methods is shown in Figure 2-11. The methods have been formulated based on different earthquake data, thus selection of which method to use should be determined from the careful evaluation of characteristics of the earthquake record and site conditions under consideration (Cai and Bathurst, 1996). An important assumption in this paper is that the base acceleration at each geosynthetic elevation is the same as the base acceleration at the ground surface. However, this is an inaccurate assumption and would result in errors as the response acceleration may be amplified with elevation especially for high walls (Cai and Bathurst, 1996). As a result, it was noted that the values of permanent displacement given by each method are only order-of-magnitude estimates rather than accurate predictions.
2.2.3.2 Internal Stability

Internal stability design considerations for seismic loading are similar to the static design considerations. The reinforcement pullout, rupture and connection failure modes as well as the internal sliding failure mode are considered. The effect of seismic loading is included in the calculations of the design tensile force where, the horizontal component of the seismic soil pressure increment and the horizontal loads due to inertial forces of the wall and structures affected by the GRS wall is taken into account.

From the NZ guidelines (Murashev, 2003):

\[ F_j^* = (\sigma_{hj}^* + \sigma_{hj,AE}^* + \sigma_{hj,q}^*) S_{vj} \]  

(2-5)

Where:
- \( \sigma_{hj}^* \) is the horizontal component of factored soil pressure from static loads;
- \( \sigma_{hj,AE}^* \) is the horizontal component of the factored seismic soil pressure increment;
- \( \sigma_{hj,q}^* \) is the factored horizontal pressure resulting from seismic lateral forces applied at the top of the wall;
- \( F_j^* \) is the design tensile force in the j-th layer of reinforcement;
- \( S_{vj} \) is the contributory area of the j-th layer of geosynthetic reinforcement.

The horizontal component of the unfactored seismic soil pressure increment is calculated as follows (NCMA 1998):

\[ \sigma_{hj,AE}^* = (0.8 - 0.6 \frac{Z_j}{H}) \Delta K_{AE} \gamma_r H \cos \beta \]  

(2-6)
Where: $z_j$ is the depth of the j-th layer of reinforcement measured from the wall crest; $H$ is the height of the wall; $\gamma_r$ is the unit weight of the reinforced backfill; $\Delta K_{AE}$ is the seismic soil pressure coefficient increment. $\beta$ is the backfill slope angle.

It is noted that the design tensile force, $F_j^*$ for a heavy facing wall should be further increased by adding the factored inertia force due to the weight of the facing element.

Analysis of the internal stability for reinforcement pullout, reinforcement rupture and connection failure under seismic conditions is similar to the static design. In the internal sliding failure mode analysis, the driving forces (inertial forces and dynamic soil force increment) is recommended to be calculated in a similar method for external stability analyses but only over the portion of the GRS wall above the sliding failure surface considered.

### 2.3 Seismic Behaviour of GRS walls in previous studies

This section summarizes the influence of GRS wall parameters such as length of reinforcement, density of backfill, wall inclination, reinforcement stiffness and others, on the seismic response of GRS walls in terms of lateral displacement, reinforcement loads, earth pressures, toe loads, acceleration amplifications, wall critical acceleration and failure mechanisms. The influence of different wall facing types is also discussed briefly. Understanding of the influences of these parameters is achieved through a literature review of GRS wall studies. The ‘bold’ key words in the rest of the section highlight the specific GRS wall parameter being discussed.

A majority of GRS wall model studies have been conducted on reduced-scale wall models under 1-g conditions. Only a handful of centrifuge GRS wall model studies have been performed such as Siddhartan et al. (2004), Nova-Roessig and Sitar (2008) and Izawa and Kuwano (2008). One of the advantages of centrifuge models is the attainment of prototype soil stress levels in the backfill. As a result, the centrifuge wall models generally shown more ductile behaviour (Nova-Roessig and Sitar, 2006) compared to 1-g wall models. Centrifuge models studies on FHR GRS walls were unable to be found in the literature with segmental retaining walls typically tested in the centrifuge model studies (Izawa and Kuwano, 2008; Siddhartan et al., 2004). As a result, due to the different facing type used, seismic responses such as wall lateral deformation, reinforcement loads and failure mechanisms are difficult to be compared between centrifuge tested wall models and 1-g reduced-scale wall models.

#### 2.3.1 Wall lateral deformation

The deformation of a GRS wall would affect its post-earthquake serviceability and therefore, GRS walls should be designed for low permanent deformations under strong shaking. Lateral displacement of the GRS wall provides a useful indication of retaining wall performance.

In GRS wall design, the geogrid reinforcement length is typically presented as a ratio to the height of the GRS wall facing, termed the reinforcement length ratio, $L/H$, where $H$ is the height of the GRS wall and $L$ the length of reinforcement. Matsuo et al. (1998) performed reduced-scale shake-table tests on discrete panel GRS wall models and showed that increasing the reinforcement length ratio $L/H$ from 0.4 to 0.7 was more effective method for reducing wall deformation compared to increasing the wall inclination or facing type. The effect of decreasing reinforcement L/H ratio leading to an increase in wall displacement is confirmed in other model...
studies (Jackson, 2010, El-Emam and Bathurst, 2007). El-Emam and Bathurst (2007) had found that decreasing the L/H ratio by 40% (from L/H = 1 to L/H = 0.6) resulted in an increase of maximum lateral displacement by about 30% at 0.32g base acceleration (Figure 2-12). In El-Emam and Bathurst (2007) and Matsuo et al. (1998), each wall model was subjected to a stepped sinusoidal excitation of 5Hz with acceleration amplitude increased in 0.05g increments of 5 s and 4 s duration respectively. A similar model excitation configuration was used in Jackson (2010) but with a longer duration of 10s.

El-Emam & Bathurst (2007) tested reduced-scale 1-g GRS wall models and reported a 25% increase in number of reinforcement layers resulted in a 36% reduction in maximum lateral displacement at an input base acceleration of 0.32g. This observation agrees well with Sakaguchi et al. (1992) which reported a 40% reduction in lateral displacement at 0.3g base input acceleration by reducing the reinforcement layer spacing by 50% (doubling the number of layers). Bathurst & Hatami (1999) also reported similar results in their numerical model (32% reduction of wall top lateral displacement by doubling the number of reinforcement layers for a 3-m high numerical wall model).

The layout of reinforcement in the GRS wall also influences its deformation behaviour. Watanabe et al. (2003) tested three wall models (reducing their size to one-tenth prototype scale) with different reinforcement arrangements: (i) Model-R1 with identical reinforcement lengths of L/H = 0.4; (ii) Model-R2 where the top and fourth layers of reinforcement had a larger length (L/H = 1.6 and 0.9) compared to the other layers of reinforcement (L/H = 0.4); (iii) Model-R3 with identical reinforcement lengths of L/H = 0.7. It was found that in terms of wall displacement, Model-R2 showed similar behaviour to Model-R3 although the total reinforcement length in Model-R2 was 20% less than Model-R3. Similar results are shown in model tests conducted by Koseki et al. (1998).

Figure 2-12: Influence of reinforcement properties on wall displacement at top of facing panel versus peak input base acceleration. Note: w = width of facing, S_v = reinforcement spacing, L/H = ratio of reinforcement length to wall height and J_m = model reinforcement stiffness (El-Emam and Bathurst, 2007).
A scaled-down recorded N-S component during the 1995 Kobe earthquake was used as the base acceleration for the wall models tested by Watanabe et al. (2003). The maximum amplitude of the base acceleration was initially set at 0.1g and incrementally increased by 0.1g until wall displacements were significant. On the other hand, a stepped sinusoidal excitation of 5 Hz and 10 s duration with acceleration amplitude increased in 0.05g increments was used by Koseki et al. (1998).

Researchers have found that the ultimate tensile strength of the reinforcement is not a determinant parameter in seismic performance of the wall, and cannot be used to predict the probable failure mechanisms and deformation modes (Sabermahani et al., 2009). Therefore, GRS wall model studies have focused on the stiffness of the reinforcement rather than the ultimate tensile strength. Model studies on the influence of reinforcement properties on the GRS wall performance have indicated that the reinforcement stiffness has a positive influence on the deformation characteristics of the GRS wall (El-Emam and Bathurst, 2007; Sabermahani et al., 2009).

Sabermahani et al. (2009) found that application of reinforcement with high tensile stiffness does not increase $G_{\text{global}}$ (the global stiffness of the backfill) greatly as compared to the influence of increasing reinforcement length. An important implication of the above researches to design is that increasing the reinforcement length and/or decreasing the reinforcement vertical spacing (increase number of layers of reinforcement) is more effective in reducing the lateral displacement of walls than increasing reinforcement stiffness.

The effect of wall inclination on the deformation of the wall was not clear from results obtained by Matsuo et al. (1998) for discrete panel facings. They found similar wall displacement in both the vertical and inclined wall models (approximately 80° to horizontal) with the inclined model having a greater displacement at mid-height but a smaller displacement at the top of the wall compared to the vertical wall. Jackson (2010) conducted model studies on FHR wall models and found clear evidence that inclined model walls results in a reduced wall lateral deformation. Similar results were obtained by El-Emam & Bathurst (2005) in their study of wall facing contribution to seismic response of model walls. They found that the inclined wall models for both hinged and sliding conditions show lower displacement values at all base input acceleration values. This is in agreement with the pseudo-static earth pressure theory that predicts a decrease in lateral earth force with increasing wall inclination into the backfill.

El-Emam & Bathurst (2005) also investigated the facing contribution to the seismic response of reduced-scale walls. The thickness (or weight) of the wall was found to be directly proportional to the top lateral displacement of the wall (ie. thicker facing panels resulted in larger top lateral displacement). This was expected as it was attributed to the larger destabilizing inertial force resulting from a larger facing mass under dynamic loading.

Latha & Krishna (2008) performed a series of 1-g shake table tests on three different wall models constructed up to a wall height of 600mm at five different backfill densities. They found that lateral wall displacement reduces with increase in backfill relative density irrespective of facing type and geosynthetic reinforcement. The lateral displacement for the wrap-faced retaining wall at 37% relative density was 2.73mm which reduced to 0.55mm at a relative density of 87%.

In terms of wall facing type, facing deformations for wrap-around walls were found to be relatively very high compared to rigid face retaining walls as shown in Figure 2-13 from Latha & Krishna’s
(2008) study. This was due to the settlement of the backfill soil during cyclic loading which decreased the radius of the wrapped face causing the face to bulge outwards as the length of geotextiles that created the wrapped face remained unchanged. Larger settlements occurred at the wall crest in the reinforced zone compared to the backfill, due to the wrap face deformations.

### 2.3.2 Reinforcement loads

To maintain the internal stability of the GRS wall, the reinforcement tensile loads experienced during periods of intense shaking must not exceed the tensile strength of the reinforcement as it would lead to reinforcement rupture and possible failure of the wall. Reinforcement loads in model studies are typically measured using strain gauges attached along the longitudinal members of the geogrid. However, it is important to note that the reinforcement tensile loads recorded are a function of the type of reinforcement used as well as the soil-reinforcement interaction. The scaling of these two properties of the reinforcement for reduced-scale 1-g model studies is difficult, so qualitative analyses are performed when investigating reinforcement loads. El-Emam & Bathurst (2004) add that the results of the connection loads for the top reinforcement layer in reduced-scale wall models must be treated with caution as the low confining stresses may have resulted in the slip of reinforcement during base excitation.

The general results from reduced-scale 1-g model studies showed an almost simultaneous increase in tensile forces in all reinforcement layers to resist the overturning moment generated when the inertia force was oriented outwards (Watanabe et al., 2003; El-Emam and Bathurst, 2007).

In their study, Watanabe et al. (2003) compared the tensile loads measured in the reinforcement for the three wall models with different reinforcement layouts. The tensile force for the top layer of reinforcement in model-R2 (where the top (800mm) and fourth (450mm) layers were longer compared to the other reinforcement layers (200mm)) was mobilized when the top displacement was relatively small (3mm) compared to a mobilization of tensile force at a top displacement of 20mm for the wall model-R1 (which had identical reinforcement lengths of 200mm for all layers). The increase or mobilization of tensile force in the top reinforcement for model-R2 could be due to

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**Figure 2-13**: Displacement profiles at the end of 20 cycles of dynamic motion: (a) wrap-faced walls, (b) rigid-faced unreinforced walls and (c) rigid-faced reinforced walls. Note that RD = relative density of backfill. (Latha and Krishna, 2008).
the resistance of the reinforcement layer to the formation of a failure plane. The top reinforcement prevented the failure plane from propagating up into the surface of the backfill from its point of origin, which was at the ends of the shorter reinforcement layers below. The study indicates that different degrees of mobilization of tensile force may be linked with the locations of the reinforcement relative to the failure planes formed.

Watanabe et al. (2003) observed a larger tensile force (12N) for the top layer of wall model-R2 which had an extended length compared to top reinforcement tensile forces in the other wall models (R3=5 N & R1= 4 N) at top displacements of 40mm. The extended reinforcement accumulates a larger tensile force due to its longer embedment length in the soil. In the wall model-R2, the fourth reinforcement layer experienced lower tensile loads when compared to reinforcement layers at identical depths in other wall models although it was longer in length (450mm compared with 350mm). There is a concentration of tensile loads in the extended uppermost reinforcement for the model wall-R2. This shows that the reinforcement layer where the largest tensile force would be mobilized may depend on the reinforcement arrangement as magnitude of tensile loads would differ for reinforcements of different lengths (Watanabe et al.; 2003)

The influence of uniform reinforcement length on the reinforcement connection loads between the reinforcement and the facing was investigated by El-Emam & Bathurst (2007) through strain gauges bonded to reinforcement that were located close to the facing. Two models of differing reinforcement length ratio, L/H of 1.0 and 0.6 were tested and the difference in reinforcement connection loads was 0.15 kN/m at 0.1g to 0.2g base accelerations which increased to approximately 0.7 kN/m at 0.4g base acceleration. Results show that the wall with the longer reinforcement length had lower reinforcement loads with the difference more pronounced in the bottom layers of reinforcement and at larger base input accelerations. This may be due to the difference in magnitude of lateral displacement between the two walls with larger lateral displacement leading to larger reinforcement loads (El-Emam and Bathurst, 2007). However, results in Watanabe et al. (2003) showed that model-R3 which had a longer reinforcement length had larger reinforcement loads at the uppermost and middle reinforcement layers despite having a lower magnitude of lateral displacement. This contradiction in results could be due to the difference in the model setup as the Watanabe et al. (2003) model had a free-sliding hinged toe whereas the El-Emam & Bathurst (2007) model had a fixed hinged toe.

El-Emam & Bathurst (2007) also observed a redistribution of reinforcement connection loads towards the bottom at base accelerations greater than 0.2g when the reinforcement vertical spacing was increased from 0.185m to 0.225m. This could be due to the higher lateral displacement experienced by the model wall that had the largest vertical spacing which led to a formation of a larger failure wedge. As a result, the anchorage length of the top layers of reinforcement was decreased, leading to increased loads at the bottom layer (El-Emam and Bathurst, 2007). Redistribution of reinforcement connection loads was observed in Watanabe et al. (2003) to a certain degree. The wall model-R1 with shorter reinforcement lengths showed a greater redistribution of tensile forces towards the lowest reinforcement layer with increasing wall displacement. The model wall with larger vertical spacing and short reinforcement layers resulted in smaller total connection loads at all stages of testing. The lateral forces acting on the GRS wall was experienced at the toe of the model wall, due to less reinforcement in the wall resulting to less total connection loads (El-Emam & Bathurst, 2007).
El-Emam & Bathurst (2007) also investigated the influence of reinforcement stiffness on the total reinforcement connection loads and found it to be larger for the model wall with stiffer reinforcement layers. This observation is in agreement with the numerical analysis conducted by Bathurst & Hatami (1998) that showed that tensile load in all reinforcement layers increases as reinforcement stiffness increases. A stiffer reinforcement increases its inextensibility, therefore reinforcement deformation is much less than soil deformation resulting in more stresses on reinforcement thus larger reinforcement loads.

The effect of reducing wall facing inclination to reduce the wall deformations also leads to a reduction in the reinforcement connection loads due to less mobilised loads (El-Emam and Bathurst, 2005). It was also observed that for a vertical wall with a thick or heavier facing, the distribution of connection loads progressively becomes more triangular with base excitation, with progressively lower connection loads near the top of the wall and higher connection loads near the bottom. The thin vertical wall shows a triangular distribution of connection loads at all levels of base excitation. Investigations conducted by Allen & Bathurst (2002) on 16 full-scale GRS wall case histories concluded that very stiff facings (including FHR and modular block/segmental wall systems) could reduce the reinforcement loads by a factor of two relative to reinforcement loads in GRS walls with flexible facings (ie. wrapped-around facing). Furthermore, El-Emam & Bathurst (2005) also showed that a horizontally unrestrained toe boundary condition results in the lowermost layer of reinforcement having the largest connection load in the wall compared to an identical wall with a hinged toe.

These case studies show that the formation of reinforcement tensile loads are complex and are due to a combination of several interdependent parameters. The wall lateral displacement is assumed to increase reinforcement loads due to larger relative movement but at the same time, the formation of a failure wedge due to lateral displacement decreases the embedment length and subsequent reinforcement loads. The model studies have also shown that the fixity of the toe of the GRS wall plays a major part in controlling reinforcement loads.

2.3.3 Wall toe loads
The use of geosynthetics as reinforcement in retaining walls reduces the need for concrete and foundations necessary compared to a typical cantilever retaining wall. GRS wall foundations or toes have a similar function to retaining wall foundations which are designed to resist lateral sliding of the wall. However, under seismic loading, some lateral movement of the foundation still could occur due to the additional forces exerted on the wall facing. Not all model studies conducted have fixed the toe of the wall due to complexity of experimental setup; Watanabe et al. (2003), Koseki et al. (1998), Sakaguchi (1996) and Jackson (2010) all performed reduced-scale model studies with a free sliding wall toe on subsoil or the model base. The model studies by El-Emam & Bathurst (2005; 2007) measured wall toe loads by fixing the toe in the lateral direction but enabling rotation of the wall about the toe. Vertical and horizontal load cells were installed at the base of the facing panel to measure the forces transmitted to the toe of the wall. Further details on the experimental setup are supplied in a separate paper (El-Emam and Bathurst, 2004).

El-Emam & Bathurst (2007) observed that the vertical toe load of the wall model was significantly greater than the self-weight of the facing panel due to vertical drag-down forces developed in the reinforcement connections during base shaking. The researchers also computed the values of the
vertical toe load predicted by a pseudo-static seismic design method (NCMA, 1998) which were found to consistently underestimate the measured vertical toe loads in all cases. The difference is due to the static and dynamic limit-equilibrium method’s inability to fully capture the drag-down forces developed in the back of the facing. The researchers found that the wall model with the greater displacements showed a larger vertical toe load, due to a larger failure wedge acting down on the connections as wall displacement increased. The inclined GRS wall exhibited a smaller vertical toe load as a portion of the self-weight carried by the soil due to its inclined nature. El-Emam & Bathurst (2005) also concluded that the toe restraint condition, be it hinged or sliding, had a negligible effect on vertical toe loads.

In El-Emam & Bathurst (2005; 2007), the restrained toe was found to attract a significant portion of the force from horizontal earth pressure acting on the facing panel at the release of the external supports after construction (30-60%) (Figure 2-14). This finding is in agreement with full-scale test wall studies (Allen and Bathurst, 2001; Allen and Bathurst, 2002). Under base excitation, the wall models carried a proportion of horizontal loads equal or slightly larger than the proportion under static conditions. EL-Emam & Bathurst (2005) found that for vertical walls, the proportion of horizontal toe load at prop release and maximum base excitation did not vary significantly (maintained at ~45%) as the geosynthetic reinforcement took up the additional peak earth pressures generated during shaking.

Figure 2-14: Horizontal toe load normalised to the total horizontal earth force for model walls with different reinforcement parameters versus peak input base acceleration amplitude (El-Emam and Bathurst, 2007).
El-Emam & Bathurst (2007) found that for the wall constructed at a longer reinforcement length of L/H of 1.0, the toe experienced a declining proportion of total force from horizontal earth pressure with increasing base input acceleration (50% at static conditions to 30% at 0.5g) compared to the model with L/H of 0.6 which had a constant horizontal toe load. This can be attributed to better reinforcement anchorage capacity mobilized during wall shaking for the longer reinforcements leading to an increase in proportion of reinforcement loads.

Their experiments also showed that the wall with greater reinforcement stiffness generated larger horizontal toe loads (15% increase as a portion of total loads for stiffer reinforcement, 90 kN/m to 1250kN/m) at all stages of testing. This is because the wall with lower reinforcement stiffness allowed more soil strength to be mobilized through greater wall deformation, thus resulting in reduced wall loading and lesser load recorded at the horizontal toe. This is consistent with the findings of Saunders and Bathurst (2002) that showed the restrained toe of a model wall with wire mesh reinforcement attracted a greater portion of the force from horizontal earth pressure than the model wall with geosynthetic reinforcement.

2.3.4 Earth pressure

Earth pressures in the GRS wall setup that are of interest are the vertical earth pressures acting below the wall foundation and the horizontal earth pressures, either acting at the wall facing or at the rear face of the reinforced backfill. Earth pressures acting on retaining walls increase with base shaking. Note that the earth pressure measured behind the facings is not equivalent to the earth pressure derived in pseudo-static analysis which is for a rigid block assumption.

In terms of horizontal earth pressures, Matsuo et al. (1998) recorded the lateral earth pressures acting on the wall facing and at the back of the reinforced soil block. The researchers found that the segmental or discrete panel facings experienced a significantly greater concentration of earth pressure at the bottom of the wall panels, where the slip surface had formed. A less significant concentration of earth pressure at the bottom of the interface between the reinforced soil block and unreinforced backfill was experienced. In contrast, the rigid facing wall experienced an increase in earth pressure that was relatively constant with depth but a greater total earth pressure at the wall facing was also experienced. This observation indicated that the formation of a slip surface at a specific location on the wall facing will increase the horizontal earth pressures at that location due to sliding of the wedge along the slip surface. The effect of inclination on the horizontal earth pressures is unclear as Matsuo et al. (1998) found a similar earth pressure concentration at the bottom of the inclined discrete facing wall as with the vertical discrete facing wall.

A concentration of vertical earth pressure was observed just beneath the toe of the wall for the segmental facing wall (Matsuo, Tsutsumi, Yokoyama and Saito, 1998). This was similar to the increase in vertical toe load discussed in the previous section and is due to settlement of the reinforced soil block increasing the vertical component of the reinforcement connection loads. This stress concentration was not observed for the rigid and inclined facing walls. A concentration of vertical earth pressure beneath the toe of the wall facing was also observed in large-scale shaking table tests on modular-block reinforced soil retaining walls (Ling, Mohri, Leshchinsky, Burkek, Matsushima and Liu, 2005). The researchers did not obtain a consistent horizontal pressure distribution for the three walls tested and concluded that the pressure distribution was likely to be affected by the degree of compaction of the reinforced backfill.
The effect of **backfill density** on the horizontal earth pressures was investigated in reduced-scale shake-table tests (Latha and Krishna, 2008). However, due to very low pressure levels with respect of the measuring range of the pressure sensors, the trend in pressure distribution along the height of the wall is inconsistent among the model walls. The effect of relative density of the backfill soil is also inconclusive. More tests to determine the effect of backfill relative density on earth pressures are needed.

### 2.3.5 Acceleration amplification

In their model studies of segmental panel GRS walls, Matsuo et al. (1998) found that acceleration amplification occurred in the reinforced backfill when the base input acceleration is relatively small (approximately 0.2g). However, at higher base acceleration (approximately 0.4g), the wall model developed noticeable plastic deformation resulting in the attenuation of input acceleration in the backward direction. Nova-Roessig & Sitar (2006) conducting centrifuge tests on wrap-around GRS walls with differing backfill densities also found similar results with the **crossover point between amplification and attenuation** at an input acceleration of 0.46g.

Nova-Roessig & Sitar (2006) found that acceleration amplification was not strongly influenced by the **reinforcement properties** (length and stiffness) or facing inclination (Figure 2-15). However, these findings conflicted with that of El-Emam & Bathurst (2007). El-Emam & Bathurst (2007) showed that for accelerations beyond the critical acceleration value, the amplification factor increased with increasing reinforcement spacing and decreased with increasing reinforcement length and increasing reinforcement stiffness. The results show that an increase in the global stiffness of the walls (i.e. sum of the layer stiffness values divided by the height of the wall) led to a decrease in the magnitude of acceleration amplification factors.

It should be noted that El-Emam & Bathurst (2004) observed that the amplification factors at the same base excitation for the top of the wall face are larger (by 0.5) than the amplifications factor at the top of the backfill. This is attributed to a phase difference of 9° between the two acceleration responses at maximum base input acceleration. This phase difference was observed to be small prior
to the critical acceleration value and significantly increased after. In a full-scale wall, out-of-phase
deformation of the rigid wall facing and the backfill soil could lead to additional reinforcement
connection loads that cannot be accounted for using current pseudo-static methods.

El-Emam & Bathurst (2005) investigated the wall facing contribution to acceleration amplification.
Amplification factors for both facing panel and backfill soil was found to be non-linear with the non-
linearity more pronounced at the facing panel for the wall models with a restrained toe compared to
a sliding toe. This is due to higher velocities at the wall crest as the facing panel oscillated about the
restrained toe. A clear trend of increased facing amplification factors with mass of the facing and
decreased amplification with increased facing inclination angle was observed.

Latha and Krishna (2008) performed shake-table tests at two different levels of dynamic excitation
and found no consistent trend regarding the influence of backfill density on the acceleration
amplification and soil pressure in all three wall models when tested at a low dynamic excitation (1 Hz
and 0.1g). However, at a higher dynamic excitation (3 Hz and 0.2g), they observed a trend of
increasing acceleration amplification with larger backfill relative density. An increase of acceleration
amplification at the top from 1.71 to 1.91 corresponded to backfill densities of 37% to 87%. Similar
results were reported by Nova-Roessig and Sitar (2006) in their centrifuge model studies.

2.3.6 Critical Acceleration

El-Emam & Bathurst (2007) investigated the influence of reinforcement parameters on the
earthquake response of FHR GRS retaining walls in their reduced-scale wall model shake-table
experiments. In their experiments, critical acceleration was taken as the point corresponding to a
sharp increase in the slope of the displacement-acceleration plots. They observed the critical
acceleration to be greater for walls with longer reinforcement as the displacement-acceleration
curves are shallower. Similar results were observed by Watanabe et al. (2003). El-Emam & Bathurst
(2007) also found that decreasing the number of reinforcement layers led to a 15% decrease in the
critical acceleration of the model wall. The effect of reinforcement stiffness on the displacement-acceleration
curve was investigated by El-Emam & Bathurst (2007) to a limited extent, up to a base
acceleration of 0.24g. Based on the limited data available, it appeared that the wall with higher
reinforcement stiffness generated a shallower displacement-acceleration curve which corresponded
to a smaller critical acceleration compared to the wall with lower stiffness.

In their model studies of FHR GRS walls, El-Emam & Bathurst (2005) showed that the critical
acceleration increased for inclined wall model and for the model wall with a lower facing thickness
(i.e. mass). The model wall with double the facing thickness of another model wall had a smaller
critical acceleration by about 0.05g, and the inclined wall model with facing inclination of 10°
resulted in an increase in critical acceleration of about 0.03g compared to a vertical wall model. Toe
fixity was also found to affect the critical acceleration of the wall model with wall models with a
sliding toe showing higher critical acceleration values compared to identical wall models with a
hinged toe. In terms of wall facing type, Ramakrishnan et al. (1998) carrying out shake table tests on
reduced scale models of segmental walls and wrap-around facing walls, showed that segmental
retaining walls can sustain approximately two times the critical acceleration of the wrap-faced wall.

Nova-Roessig & Sitar (2006), defining critical acceleration as the minimum acceleration that causes
the slope face to deform permanently, found that the observed critical acceleration was a stronger
function of the backfill density than of the peak base input acceleration (Figure 2-16). An increase in
observed critical acceleration of 121% was observed as Dr increased from 74% to 90%, whereas an increase of only 39% was observed when peak base input acceleration increased from 0.2g to 0.9g. Increase in observed critical accelerations with increasing base input acceleration was attributed to the more pronounced inward displacements (due to strong outward accelerations) under stronger intensity shaking. This indicated a strong influence of backfill density on the stability of the GRS wall and the need for a good degree of compaction in practice.

Note that the displacement-acceleration curves where a sharp increase in the slope of the curve beyond the critical acceleration value observed in the 1-g reduced scale FHR wall model tests in El-Emam and Bathurst (2007) and Jackson (2010) are not observed for the centrifuge model studies. Centrifuge model studies by Siddharthan et al. (2004) and Izawa and Kuwano (2008) show that a sudden increase of maximum wall displacement was observed early in initial stages of model excitation and a linear relationship between wall displacement and acceleration developing in the subsequent excitation levels. This is likely due to the relatively high deformability of wall-around facings (due to settlement of the backfill soil) compared to FHR facings.

Due to the lack of centrifuge wall model studies on FHR walls, we are unable to verify the occurrence of a critical acceleration value for FHR wall models. However, centrifuge wall model studies on conventional retaining walls by Okamura et al. (2003) show displacement-acceleration curves with a clear critical acceleration threshold similar to 1-g reduced scale FHR wall model studies. This indicates a likelihood that FHR GRS wall models tested in a centrifuge will have similar displacement-acceleration curves and thus a critical acceleration threshold beyond which a sharp increase in wall displacement is experienced.
2.3.7 Observed failure mechanisms

Watanabe et al. (2003) carried out shake-table tests on FHR GRS walls with different reinforcement layouts. As shown in Figure 2-17, no failure planes were observed at the bottom of the front wedge in the reinforced zone which does not agree with the two-wedge failure mechanism (Horii et al., 1994) assumed in current design procedure in Japan. However, the front wedge did not behave like a rigid body and suffered simple shear deformation along the horizontal planes. This indicates that the failure plane forms with difficulty through the reinforced backfill and the horizontal reinforcement is unable to resist simple shear deformation of the reinforced backfill.

Figure 2-17: Residual displacement of 6 types of retaining walls observed after the final shaking step (Watanabe et al., 2003).
An extended reinforcement length for the top reinforcement layer of a GRS wall was shown by Watanabe et al. (2003) to affect the formation of failure mechanisms in a GRS wall. The formation of two failure planes of similar angles (~41°) occurred simultaneously. The upper failure plane extended from the back of the reinforced zone (beside the extended 4th reinforcement layer) and ended somewhere just below the extended top layer of reinforcement and the lower failure plane formed just beside the end of the top layer of reinforcement. This indicates that the location of the failure planes is strongly governed by the existence of the extended reinforcement.

Jackson (2010) conducting reduced-scale model tests on FHR GRS walls, observed a progression of deformation with increasing base excitation. The application of higher accelerations caused the progressive formation of deeper inclined failure surfaces with all failure surfaces at a similar angle to the horizontal. When the critical acceleration of the wall model was reached, the failure surface propagated along the lowest reinforcement layer due to confinement of the soil by the rigid foundation (box base). The GeoPIV analysis of the reinforced backfill shows that both simple shear and localised strain deformation occurred along the horizontal planes of the reinforced backfill. These findings are consistent with that of Watanabe et al. (2003) as the failure plane would form with difficulty through the reinforced backfill and simple shear deformation would occur within the reinforced backfill.

El-Emam & Bathurst (2005) concluded that for each model wall regardless of facing thickness (mass) or inclination, the predominant failure mode was wall rotation about the toe. However, beyond the critical acceleration, base sliding became the dominant wall deformation. This agrees with results from Jackson (2010), where tests on FHR GRS reduced-scaled walls found for all base input accelerations, the predominant component of deformation was rotation with the sliding component only becoming significant at the final shaking step of the wall at failure.

Note that in centrifuge model studies by Izawa and Kuwano (2008), the failure surface observed in the test developed through the reinforced backfill at an inclined angle and propagated upwards to the backfill surface. Due to the segmental wall facing used in their models, Izawa and Kuwano (2008) showed that base sliding was the dominant wall deformation mode. Similar observations can be made for results from Nova-Roessig and Sitar (2008) in their centrifuge model studies of wrap-around reinforced soil walls. Maximum deformation in both centrifuge wall model studies occurred at about mid-height of the wall models (likely due to the relative deformability of wall facings used).

### 2.4 Accuracy of design guidelines in predicting behaviour of GRS walls

El-Emam & Bathurst (2005; 2007) investigated the accuracy of current pseudo-static methods to predict reinforcement loads for internal stability of GRS walls. The NCMA method predicts connection loads using the M-O method with the contributory area approach (based on tributary areas of each layer of reinforcement), peak soil friction angle and the assumption of fully-mobilized soil-wall friction angle. The current AASHTO/FHWA guidelines use a different procedure to calculate reinforcement connection loads where the value and distribution of the dynamic earth force increment is weighted, based on the total anchorage length embedded in the resistance zone. The failure wedge geometry is defined using Rankine theory and is assumed to propagate from the heel of the facing column at an angle $\alpha = \pi/4 + \phi_{peak}/2$. In the AASHTO/FHWA method, the weight of
the facing is not taken into account in the dynamic earth force calculations. The NZ design guidelines by Murashev (2003) are based on the NCMA design guidelines.

Both guidelines were found to overestimate the **reinforcement loads at the top of the wall and underestimate the reinforcement loads at the bottom of the wall** regardless of wall type and toe boundary condition (El-Emam and Bathurst, 2007). This over/underestimation of reinforcement load increased with base input acceleration, with the NCMA method being progressively less accurate and more uniformly distributed than the measured values with increasing base acceleration. The poor agreement could be attributed to the assumed magnitude and shape of the dynamic portion of soil earth pressure assumed in the pseudo-static method. The assumption that the dynamic load increment acts at 0.6H above the toe led to a larger portion of incremental dynamic force being taken up by the top layers of reinforcement in the design calculations. The AASHTO/FHWA method was better able to predict the distribution of load with increasing base excitation for hinged toe models.

In El-Emam & Bathurst (2007), the researchers found that for the model walls with smaller reinforcement spacing ($S_v = 185\text{mm}$), the **increase of total reinforcement load with base excitation** was more closely predicted by the NCMA method (steeper increase in total reinforcement loads with base excitation) compared to the AASHTO/FHWA method. The AASHTO/FHWA method was in agreement with the model wall with larger reinforcement spacing ($S_v = 225\text{mm}$). The same was found for reinforcement of differing reinforcement stiffness with the NCMA-predicted trend of total connection loads closer to the response curve for the stiffer reinforcement ($J_m = 1250\text{kN/m}$) and AASHTO/FHWA method more accurately predicting the trend of the model wall with low reinforcement stiffness ($J_m = 90\text{kN/m}$).

For the model walls with a restrained toe, NCMA and AASHTO/FHWA methods were both found to underestimate the **total earth force acting on the wall** (connection load + toe load) as shown in El-Emam & Bathurst (2007). However, the actual response trend of the model walls was better estimated by the NCMA approach. This showed that the pseudo-static methods were non-conservative for the design of a typical restrained wall toe. The pseudo-static design approach also consistently underestimated the **vertical toe loads** as it neglected the vertical drag-down forces that developed at the back of the facing as a result of wall rotation and lateral deformation. This underestimation increased at higher input base accelerations (El-Emam and Bathurst, 2005).

The NCMA and AASHTO/FHWA guidelines use the equation proposed by Segrestin & Bastick (1988) that relates the horizontal acceleration coefficient, $k_h$, to the PGA, $\alpha_g$. However, the design guidelines predicted an opposite trend of **acceleration amplification** compared to experimental data for high base accelerations. The experimental data from model studies by El-Emam and Bathurst (2007) showed that acceleration amplification increases with base input acceleration but the predicted amplification factors from current NCMA and AASHTO/FHWA guidelines decreased with base input acceleration. Below a base acceleration of 0.3g (maximum value for pseudo-static design methods), there was a reasonably good agreement with the experimental data from El-Emam and Bathurst (2007). However, results from centrifuge studies of wrap-around GRS wall models by Nova-Roessig and Sitar (2006) show good agreement with predicted amplification factors from the guidelines with a decrease of amplification factors from 1.5 (at 0.1g) to 1.05 (at 0.4g) observed.
These conflicting results demonstrate the need for further research to clarify the trend of acceleration amplification with increasing base excitation.

The recommended practice for pseudo-static design according to AASHTO/FHWA guidelines is to assume a constant amplification factor at all locations in the backfill. However, constant amplification factors was not observed in model studies (Jackson, 2010). Jackson (2010) found that the reinforced zone of the backfill experienced greater amplification by approximately 20%, compared to the unreinforced zone of the backfill.

In conclusion, the current pseudo-static methods do not accurately model the seismic behaviour of GRS walls leading to discrepancies between predicted behaviour and observed behaviour of GRS wall models tested. The current limitations of the pseudo-static design methods should be understood by the engineer to ensure a safe but economical design of the GRS wall. More studies and analyses will have to be done to produce better design methods in the future.

2.5 Summary

A review of the current design methodology for geosynthetic-reinforced soil (GRS) walls under seismic loading has been presented. Additionally, seismic design considerations such as the effect of dynamic loading on the material properties and the development of acceleration amplification and the GRS wall critical acceleration has been briefly discussed. One implication from this review is that the reinforcement stiffness and tensile working strength of the reinforcement might be underestimated in design leading to an overly conservative design.

Furthermore, findings of previous researchers with regard to the effect of GRS wall parameters on the seismic performance of the GRS wall were discussed. Several key parameters such as reinforcement length, backfill density, wall inclination and wall facing type were found to strongly impact the deformation of the GRS wall. The typical failure mechanism of a GRS wall is found to be predominantly wall rotation about the wall toe with sliding of the wall toe increasing in significance beyond the critical acceleration.

Lastly, a brief discussion of the accuracy of current design guidelines was presented. Current pseudo-static methods were found to not accurately model the seismic behaviour of GRS walls in terms of reinforcement loads, earth pressures and acceleration amplification. These limitations should be understood by the design engineer to ensure a non-overly conservative design of the GRS wall.

2.6 References


Chapter 3.0 EXPERIMENTAL DESIGN AND TESTING PROCEDURES

3.1 Introduction
In this study, twelve reduced-scale GRS wall models were tested on a shake-table under 1-g conditions. Chapter 3 presents in detail the design of the experimental model, construction methodology, instrumentation and testing procedures used. Firstly, Section 3.2 presents the model design and details, including factors under consideration when designing different aspects of the wall models such as the wall seal, backfill soil and type of reinforcement. The section also considers scaling considerations during modelling of the GRS wall model at a 1:5 scale. Next, Section 3.3 presents details regarding the shake-table used in the studies and the motion dynamics associated with the shake-table.

In Section 3.4, the instrumentation used in the GRS wall models is presented. These include accelerometers, potentiometers, load cells, earth pressure cells and high-speed cameras for GeoPIV analysis. The model construction methodology and procedures are shown in Section 3.5. These involve the initial bracing of the FHR wall panel, reinforcement layer installation and placement and the wall model deposit construction. Lastly, a summary table of the model deposit densities achieved for each wall model tested is shown in Section 3.6.

3.2 Model Design and Details
The GRS wall model used in current studies was previously designed and constructed by Jackson (2010). A prototype full-height rigid (FHR) wall was first designed using recommendations in FHWA (2001) and then scaled down by a ratio of 1:5 to give the wall model details.

Appropriate wall parameters such as wall facing, reinforcement and soil in the model were selected based on scaling rules suggested by Iai (1989) to satisfy geometric, dynamic and kinematic similitude (Jackson, 2010).

The similitude laws for the main components of GRS wall modelling are listed in Table 3-1 where \( \lambda \) is the geometric scale factor (Iai, 1989). The subscripts \( m \) and \( p \) denote the model and prototype parameters respectively. Here it is assumed that the same soil is used for the model and the prototype, so the scaling factor for density of the soil, \( \lambda_p \), is 1.

Since the major concern of dynamic GRS model tests is the deformation, rather than the ultimate stability of the soil structure, the similitude laws derived from Iai (1989) are deemed applicable for dynamic model tests. Pinto & Cousens (1999), in their attempt to accurately model geosynthetic reinforcement, conducted similarity (also based on Rocha (1957) assumptions) and dimensional analyses to determine the similitude conditions required for accurate modelling. Identical scaling factors were achieved from both analyses and these agreed with the similitude laws proposed by Iai (1989) (Pinto, et al., 1999).
Table 3-1: Summary of Scale factors for modelling geosynthetics

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Relationships(^1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stress</td>
<td>( \sigma_m = \left( \frac{1}{\lambda} \right) \sigma_p )</td>
</tr>
<tr>
<td>Elastic Modulus of Material</td>
<td>( E_m = \left( \frac{1}{\lambda^2} \right) E_p )</td>
</tr>
<tr>
<td>Strain</td>
<td>( \varepsilon_m = \varepsilon_p )</td>
</tr>
<tr>
<td>Displacement</td>
<td>( \delta_m = \left( \frac{1}{\lambda} \right) \delta_p )</td>
</tr>
<tr>
<td>Unit weight of Backfill</td>
<td>( \gamma_m = \gamma_p )</td>
</tr>
<tr>
<td>Length of Reinforcement</td>
<td>( l_m = \left( \frac{1}{\lambda} \right) l_p )</td>
</tr>
<tr>
<td>Internal friction angle of material</td>
<td>( \phi_m = \phi_p )</td>
</tr>
<tr>
<td>Stiffness of reinforcement</td>
<td>( J_m = \left( \frac{1}{\lambda^2} \right) J_p )</td>
</tr>
<tr>
<td>Particle Size</td>
<td>( d_m = \left( \frac{1}{\lambda} \right) d_p )</td>
</tr>
</tbody>
</table>

\(^1\lambda\) is the geometric scale factor

### 3.2.1 Prototype Wall design

In Jackson (2010), the GRS wall was designed using the simplified coherent gravity method (FHWA, 2001). The failure modes considered in the design were overtopping and sliding (external) and reinforcement pullout (internal). The absence of foundation sub-soil as well as the use of a FHR facing excludes failure mechanisms such as deep seated slip, bearing capacity and internal block sliding failure.

The GRS wall was designed to a prototype height of 4.5m with a typical reinforcement length to height ratio, \( L/H \) of 0.75 with 5 reinforcement layers spaced at 750mm (Jackson, 2010). It is noted that the design is well within the minimum values recommended by the FHWA (2001) which is a reinforcement ratio, \( L/H \) of 0.7 and vertical spacing of 500mm. The prototype wall detail corresponds to scaled-down wall model height of 900mm and reinforcement vertical spacing of 150mm based on a 1:5 scale.

### 3.2.1 Rigid Box design

The GRS wall model was constructed within a rigid steel box for the ease of construction, testing and de-construction of the wall models. One side of the rigid box was made of 20mm thick transparent acrylic to allow for a side view of the model and to enable high-speed digital image capture of the backfill during model excitation for subsequent GeoPIV analysis. A Perspex sheet was taped to the transparent acrylic window to protect the window from the abrasive properties of the sand backfill. After every test, the Perspex sheet was polished and cleaned to remove any minor scratches caused by the sand backfill to ensure a clear view of the backfill deformation through the transparent window was maintained.

The rigid box had the plan dimensions of 0.8m by 3m with a depth of 1.15m. Similar box dimensions were used by other GRS model studies (Sakaguchi, 1996; Watanabe et al., 2003; El-Emam and Bathurst, 2007) as seen in Table 3-2. Figure 3-1 presents a general alignment of the rigid box and the GRS wall model constructed within it for a uniform wall model of 0.75 reinforcement length ratio. The base of the rigid box was lined with plywood covered with glued sand to provide frictional resistance at the wall toe.
Table 3-2: Summary of other researcher’s and this study’s model dimensions and materials (Jackson, 2010)

<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>Current Study (Similar to Jackson, 2010)</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 sidewall (20mm thick).</td>
</tr>
</tbody>
</table>

Figure 3-1: Dimensions of Rigid box and GRS wall constructed within (Bold black line outlining the rigid box).

### 3.2.2 Wall seal

An important detail of the facing panel is the seal the GRS wall panel makes with the rigid box sidewall. The wall seal should prevent any leakage of sand around the wall face but with minimal influence on the model behaviour by generating low friction.

Initially, the existing wall seal (Seal A) fabricated by Jackson (2010) was reused for consistency of results. The existing seal design was based on experimental methods by Watanabe et al. (2003) and used foam and Teflon strips to create a ‘frictionless’ seal with respect to the wall movement. However, some leakage was occurring throughout the construction phase, leading to a collection of sand at the wall toe. At the wall toe (up to a height of 80mm), some of the sand got trapped between the Teflon sheet and the sidewall as observed in Figure 3-2(a). This would have increased the frictional resistance in that area, mostly affecting the sliding movement of the FHR wall panel. In KL2, the Seal A failed after compaction of the final top layer of sand leading to sand leakage. This was remediated by using a thin rod to press the seal back in place to prevent it from opening up throughout the test.
In the following test, KL3, the wall seal was reconfigured, keeping the same Teflon sheets but increasing the thickness and width of the foam holding the Teflon sheet to make the seal stiffer (Seal B). However, it appeared that this new wall seal setup was too strong as upon wall failure for wall model KL3, the wall was unable to fully rotate and rest on the external support.

This led to a change in the material used for the wall seal. In the new wall seal (Seal C), an ultra-high molecular weight (UHMW) polyethylene strip was used for the seal instead of the Teflon strip. This was stiffer than the Teflon strip and did not require a foam strip to support it. The dynamic coefficient of friction of the material on polished steel is 0.10-0.22 which is comparable to that of Teflon (0.04-0.25). The sidewalls that would be in contact with the seal during the test were polished beforehand to enable wall displacement to progress smoothly. Results from using Wall Seal C are shown in Figure 3-2(b).

Spring force tests were used to quantify the friction force of the wall in rotation. To do so, a spring balance was used to apply an overturning force to the top of the FHR panel. An average overturning force of 2 - 3N was determined to be the force magnitude required to overcome the frictional resistance of the wall seal and cause the wall to fail. When compared to the forces exerted by the backfill on the wall in the testing phase, the critical overturning force is very small at approximately 0.2% of theoretical total horizontal force applied to the wall under static conditions. Therefore, Seal C was deemed suitable. Furthermore, using the Seal C setup, no sand was lost due to leakage at any stage of testing. This was an improvement from Seal A where sand loss was experienced throughout the construction phase. This ensured that the density of the backfill could be determined with greater accuracy during the backfill construction and more importantly that the leaked sand was not influencing the movement of the wall. Wall Seal C was used for the wall models KL4 to KL12.

Figure 3-2: (a) Sand leakage present at the wall toe in KL2 with Wall Seal A; (b) No sand leakage present at the wall toe even after excitation till failure with Wall Seal C.
3.2.3 Soil

The same soil used by Jackson (2010) in his studies, Albany sand is used as the model soil due to its ready availability, known properties and that it generates minimal dust during deposition. The properties of Albany sand are presented in Table 3-3.

Table 3-3: Albany sand soil properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solid particle density, $\rho_s$</td>
<td>$2.65 , t/m^3$</td>
</tr>
<tr>
<td>Mean particle size, $D_{50}$</td>
<td>$0.3 , mm$</td>
</tr>
<tr>
<td>Maximum void ratio, $e_{max}$</td>
<td>0.83</td>
</tr>
<tr>
<td>Minimum void ratio, $e_{min}$</td>
<td>0.51</td>
</tr>
<tr>
<td>Peak friction angle, $\phi_p$</td>
<td>$33^\circ$</td>
</tr>
<tr>
<td>Critical state friction angle, $\phi_{ss}$</td>
<td>$31^\circ$</td>
</tr>
</tbody>
</table>

As determined by Roper (2006)

In geotechnical models, the soil stresses within the model is the dominant factor behind soil behaviour, in terms of strength, stiffness and development of strain. In reduced-scale models, the soil stresses within the model are of a reduced magnitude compared to soil stresses achieved at the prototype scale; soil stresses are not one-to-one scaled. In order to produce identical self-weight stresses in the model and prototype, many researchers such as Ling et al. (2004), Nova-Roessig and Sitar (2006) and Viswanadham and Konig (2009) have used geotechnical centrifuges in their GRS wall models.

However as current tests are conducted at 1-g, special consideration of soil density, vertical confining stresses and soil stiffness is necessary. To ensure representative model behaviour at large strains associated with model deformation, critical state soil mechanics in considered in this study (Wood, 2004). A schematic diagram detailing critical state soil mechanics and its use in experimental modelling is shown in Figure 3-3.

![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).](image-url)
To achieve similar values of the state variable, $\Psi$ and thus relatively similar soil states, along with the decrease in soil stresses in a model, a relative change in void ratio, $e$ is required. Hence for a typical GRS wall prototype (at high stresses) constructed with Albany sand at a relative density of 90%, the corresponding scaled-down wall model (at low stresses) will have to be constructed at a higher void ratio. As a result, a target backfill density of 50% was selected for a majority of wall models tested in this research. A wall model of approximately 90% relative density was also tested for comparisons with test results of Jackson (2010) who used model relative densities of 90% in his studies. Sabermahani et al. (2009) also noted that in theory the model should be looser than the prototype and tested models with similar relative densities of 47% and 84%. However, it is noted that the positive effects of decreasing the relative density of the soil material might possibly be offset by the consequent decrease in effective stress acting at the soil-geogrid interface due to the lower relative density of the material.

Besides reducing the relative density of the soil to achieve similitude, the low confining stress for the model compared to the prototype can be addressed by increasing the effective stresses acting on the soil specimen by means of a uniform surcharge. Therefore, the need for a large decrease in backfill relative density to maintain similitude between model and prototype is reduced. In the literature, sparse information is available regarding the effect of surcharging on the response of GRS walls. This is possibly because the inertial load from a soil surcharge has the potential to affect the wall loading thus undermining the reliability of the experimental results. Scaling issues of soil stresses are typically addressed by testing the wall models in a centrifuge.

### 3.2.4 Reinforcement Details

A major difficulty in model studies involving geotextiles and geogrids is the selection of geosynthetic materials. Unlike soils, the similitude condition does not allow the use of identical geosynthetic materials to be used in model and prototype studies. Viswanadham & König (2004) states that to model the geosynthetic reinforcements correctly, two similitude relationships have to be fulfilled: (1) Scaling of tensile strength-strain behaviour; (2) Scaling of frictional bond behaviour between the soil and geosynthetic.

#### 3.2.4.1 Scaling of Tensile Strength-strain behaviour

From previous studies (Sabermahani, et al., 2009), we know that the stiffness of the reinforcement is the dominant parameter for the prediction of the deformational behaviour of GRS walls compared with the ultimate tensile strength of the reinforcement. Therefore, the scaling of the reinforcement stiffness is of interest when modelling GRS walls. Scaling studies for geosynthetic reinforcement by Iai (1989) and Pinto & Cousens (1999) determined that the scaling relationship for reinforcement stiffness is:

$$J_m = \left( \frac{1}{\lambda^2} \right) J_p$$

Where: $J$ is the reinforcement stiffness; $\lambda$ is the geometric scale factor.

Jackson (2010) showed that for a typical geogrid used at the prototype scale (Tensar UX1800HS was used) which has a corresponding axial stiffness, $J_{2\%}$ of 2375 kN/m, the corresponding model stiffness is 95 kN/m.
In a recent study, Nakajima et al. (2008) performed one-tenth scale model tests of GRS FHR walls with different geogrid reinforcement stiffness of approximately 185 kN/m and 60kN/m respectively. This corresponded to the two prototype geogrid stiffness of 18,500 kN/m and 6,000kN/m according to scaling laws by Iai (1989). However, even though the material properties of the reinforcement were largely different, the models demonstrated little observed difference in seismic performance (Nakajima, et al., 2008). Conflictingly, studies conducted by El-Emam and Bathurst (2007) on one-sixth scale model GRS walls showed that GRS wall models with differing model geogrid stiffness (90 kN/m to 1250 kN/m) exhibited very different seismic behaviours (Figure 3-4). The model geogrid stiffness corresponds to prototype stiffness of 3,240 kN/m and 45,000 kN/m respectively. The Nakajima et al. (2008) study indicates that relatively small changes to the geogrid reinforcement stiffness could have a negligible impact on the seismic behaviour of the GRS wall. This is possibly due to the low influence of the change in reinforcement stiffness on the global stiffness of the GRS wall but further investigations are required to fully understand this.

Besides that, the stiffness of the geogrid plays a role in its extensibility. Reinforcement extensibility is an important reinforcement parameter that affects the formation of lateral earth pressures within the reinforced soil block. A particular geogrid may be considered an extensible reinforcement at prototype scale but will behave more like a non-extensible reinforcement due to its higher stiffness in model scale (Jackson, 2010). For example, in a model setup, a particular reinforcement is relatively much stiffer compared to the soil, resulting in a resistance to the formation of active earth pressures. However, in a prototype setup, the geogrid is at about the same stiffness compared to the soil therefore, the reinforcement will accumulate strain and this induces the active earth pressure of the wall. Therefore, care must be taken to ensure similarities in terms of reinforcement extensibility for model and prototype. If this is not accounted for, researchers should take special consideration in interpreting lateral earth pressure coefficients in the model.

![Figure 3-4: Influence of reinforcement stiffness on the measured and predicted sum of reinforcement connection loads with peak input base acceleration amplitude (El-Emam and Bathurst, 2007).](image)
3.2.4.2  Scaling of Soil-Geogrid Interaction

Scaling of the soil-geogrid bond interaction for 1-g testing has not been investigated in great detail in the literature. Papers found by the author only discuss scaling considerations for modelling geosynthetics in a centrifuge (Viswanadham, et al., 2007; Zornberg, et al., 1997).

From the dimensional analysis and similitude equations from Iai (1989) and Pinto & Cousens (1999), the interface friction angles between the soil and different materials should be the same value for the model and prototype (this is termed the law of equal interface friction in this report). This is in agreement with work done by Weber (1968) that suggested that frictional resistance is assumed to both obey Coulomb’s law and have the same internal friction angle ($\phi_m = \phi_p$) independent of normal stress and rate of shearing (Pinto and Cousens, 1999).

Studies (Gourc, et al., 1992) have reported an apparent scale effect between mobilized bond stress, $\tau_b$, and displacement between the soil and the geosynthetic reinforcement, $\delta$. This scale effect is likely due to the different soil stresses between the model and the prototype which results in different mobilized bond stresses. Viswanadham and Konig (2004) illustrate this scale effect through the idealized relationship between bond stress and displacement for the prototype 1g and $\lambda$-g tests respectively in Figure 3-5.

Studies by Viswanadham and König (2004) show that the soil-geogrid interaction is complex and current scaling laws do not account for the differences in interaction for the model and prototype. More research and possible numerical simulations are needed to enable researchers to carry out quantitative analyses of GRS wall models.

![Figure 3-5: Idealized variation of bond stress with relative displacement ($N = \lambda$) (Viswanadham and König, 2004).](image)
### Model Geogrid reinforcement used

For our model studies, the Microgrid reinforcement manufactured by Stratagrid was selected as the model reinforcement. The relevant design properties of the Microgrid are detailed in Table 3-4.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value $^1$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial stiffness, $J_{2%}$</td>
<td>220 kN/m</td>
</tr>
<tr>
<td>Ultimate tensile strength, $T_{ult}$</td>
<td>29.2 kN/m</td>
</tr>
<tr>
<td>Creep limited tensile strength</td>
<td>18.5 kN/m</td>
</tr>
<tr>
<td>Long term design tensile strength (LTDS)$^1$, $T_{at}$</td>
<td>14.0 kN/m</td>
</tr>
</tbody>
</table>

$^1$ LTDS is based on the ultimate tensile strength, $T_{ult}$ reduced by reduction factors due to creep, installation damage and durability

$^1$ Strength and stiffness denoted as per unit width

Although the axial stiffness of the Microgrid is 220 kN/m, approximately double the similitude derived stiffness of 95 kN/m derived in Section 3.2.4.1, the difference in magnitude is similar to that encountered by Nakajima et al. (2008) and thus was deemed to model the reinforcement sufficiently as little difference in seismic performance between the models was observed in their studies.

The Microgrid reinforcement stiffness is higher than that used by previous researchers, Sabermahani et al. (2009) and El-Emam and Bathurst (2004) of 0.09 – 29 kN/m and 90 kN/m. Due to its higher stiffness at model scale, the Microgrid may behave as a non-extensible reinforcement instead of an extensible reinforcement typically used in prototype walls. As a result, care must be taken when interpreting the lateral earth pressures of the wall models (Jackson, 2010).

### Backfill Surcharging

For wall models KL9, KL10, KL11 and KL12, a 3kPa surcharge was applied to the top of the backfill to increase the soil stresses within the model. This surcharge was achieved by using a combination of steel plates (35kgs each) and packed bags with steel punching (11kgs each). The hybrid combination for the surcharge was developed to ensure a fluid surcharge load that would be able to freely deform with the backfill is achieved. The initial location of the surcharge is 150mm from the wall facing and 240mm from the far-field wall. This is to ensure the surcharge does not apply any inertial load to the wall facing and interact with the far-field wall as possible out-of-phase movement of the surcharge and the rest of the model could occur during model excitation due to their different dynamic properties.

The surcharge bags were placed in between individual steel plates and between the steel plates and the rigid box side wall to prevent dynamic interaction during excitation, acting as dampers to prevent possible ‘pounding’ in both interaction cases. To limit localized shearing from occurring at contact points between the sand backfill and the surcharge, the surrogate layer were placed on top of foam-like material of 8mm thickness which was placed directly on top of the backfill. The layout of the surcharge is presented in Figure 3-7.

Using the Boussinesq’s solution (Boussinesq, 1885), we are able to determine the increase of stresses and strains with the homogeneous backfill due to the application of the surcharge. Influence factors ($I_i$) for vertical stress due to a uniform strip load were used to determine the contour plots for the increased vertical stresses due to the 3kPa surcharge (Bowles, 1997). The increase in vertical stress within the backfill due to surcharge application is presented in Figure 3-6 and show a decreasing effect of the surcharge on vertical stress with increasing depth.
3.2.5.1 Determination of Corresponding Horizontal Stresses

Due to the extra surcharge placed onto the GRS wall backfill, an increase in horizontal stresses acting to destabilize the wall is expected. With the determined increase in vertical stress from Boussinesq’s solution, the corresponding increase in horizontal stresses acting against the GRS wall can be calculated. To do so, the active earth pressure coefficient, $K_A$, is first determined from the critical state friction angle of $31^\circ$ for Albany sand determined by Roper (2006). $K_A$ is determined to be 0.32.

Theoretical static earth pressures are determined and presented in Figure 3-8 and Table 3-5. From Figure 3-8, an increase of horizontal stresses of 10-150% is expected at the wall centerline due to the 3kPa surcharge with the degree of influence of the surcharge on the horizontal stresses decreasing with depth. This indicates that the surcharge has a greater effect on the soil-geogrid interaction for the top few reinforcement layers compared to the bottom reinforcement layers. Note that this observation is important when analyzing tests results in the following chapters.
Figure 3-8: Increase in horizontal stresses at the FHR wall panel along the centreline due to surcharge application.

Table 3-5: Increase in backfill stresses along the centreline due to 3kPa surcharge application

<table>
<thead>
<tr>
<th>Depth (mm)</th>
<th>Dr=50% Deposit with no surcharge</th>
<th>Dr=50% Deposit with 3kPa surcharge</th>
<th>Increase of horizontal stress due to surcharge</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Vertical Stress (kPa)</td>
<td>Horizontal Stress (kPa)</td>
<td>Vertical Stress (kPa)</td>
</tr>
<tr>
<td>150</td>
<td>2.3</td>
<td>0.8</td>
<td>5.8</td>
</tr>
<tr>
<td>300</td>
<td>4.7</td>
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<td>7.7</td>
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<tr>
<td>450</td>
<td>7.0</td>
<td>2.2</td>
<td>9.5</td>
</tr>
<tr>
<td>600</td>
<td>9.3</td>
<td>3.0</td>
<td>11.4</td>
</tr>
<tr>
<td>750</td>
<td>11.7</td>
<td>3.7</td>
<td>13.4</td>
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<tr>
<td>900</td>
<td>14.0</td>
<td>4.5</td>
<td>15.5</td>
</tr>
</tbody>
</table>

### 3.3 Shake-Table and Motion Dynamics

The rigid box with the GRS wall braced within it was mounted onto the University of Canterbury shake-table through two series of bolts along the rigid box sides. The shake-table has two degrees of freedom, with its motion uni-directional in the horizontal direction. The shake-table is 4.0m long and 2.0m wide with a maximum velocity of 240mm/s, maximum payload of 20 T and a peak displacement amplitude stroke of ± 120mm. The shake-table system is driven by a 200kN double-acting hydraulic actuator, powered by three 100 Horsepower motors with an operating pressure of 3000 PSI (pounds per square inch). The hydraulic actuator is controlled by a set of two E072-054 servovalves controlled by a MTS FlexTest-40 digital controller. The system has a built-in LVDT (Linear Variable Differential transducer) which allows the monitoring of the shake-table displacement.

The shake-table is displacement-controlled with the displacement amplitude and frequency of movement is controlled to generate the desired acceleration as shown through the governing dynamic equation below:

\[
a(t) = -\omega^2d(t) \\
\omega = 2\pi f
\]  \hspace{1cm} (3-2)
Where $a(t)$ and $d(t)$ are the shake-table acceleration and displacement amplitudes and $\omega$ and $f$ are the angular and temporal frequencies of motion. By using a sinusoidal motion of frequency 5Hz and displacement amplitude of 1mm, a sinusoidal acceleration record with amplitude of 0.1g can be achieved by the shake-table.

Murahidy (2004) determined that the University of Canterbury shake-table has an unloaded frequency of 17.5 Hz and developed a relationship between shake-table payload and natural frequency. Using this relationship, we are able to determine the natural frequency of the shake-table, rigid box and backfill mass of the testing system. Natural frequencies of 13.7 Hz and 14.8 Hz for 90% and 50% relative density wall models are determined for the wall model system, larger than the excitation frequency of 5Hz indicating that resonance effects would not occur during excitation.

### 3.4 Instrumentation

Instrumentation of the GRS wall model consists of accelerometers, earth pressure cells, reinforcement load cells, displacement transducers and three high-speed cameras for digital image capture. Instrumentation performed in the current tests are similar to that done in Jackson (2010) but with additional accelerometers and the introduction of load cells and pressure cells. In total, the model has 9 load cells, 11 accelerometers, 4 pressure cells, 6 displacement transducers and three high-speed cameras.

An instrumentation setup for a typical GRS wall test is shown in Figure 3-9 with spatial coordinates for all instruments are listed in Table 3-6. The spatial coordinates listed are based on the point of origin located at the base of the FHR wall panel as shown in Figure 3-9.

![Figure 3-9: Instrumentation Setup of a typical GRS wall model (Uniform L/H = 0.75): (a) Plan-view; (b) Side-view.](image)
Table 3-6: Instrumentation layout for accelerometers, load cells, potentiometers and earth pressure cells within the wall model

<table>
<thead>
<tr>
<th>No.</th>
<th>Instruments</th>
<th>Groups</th>
<th>Name</th>
<th>Code</th>
<th>Channel</th>
<th>X</th>
<th>Y</th>
<th>Z</th>
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</thead>
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<td></td>
<td>Acc2</td>
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<td>680</td>
<td>225</td>
<td>400</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td>Acc4</td>
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<td>200</td>
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<td>Acc5</td>
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<td>400</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td></td>
<td></td>
<td>Acc6</td>
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<td>1610</td>
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<tr>
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<td></td>
<td></td>
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<tr>
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<td></td>
<td>Acc9</td>
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<tr>
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<td></td>
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<td>--</td>
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<td>Acc11</td>
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<tr>
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<td>150</td>
<td>399</td>
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<tr>
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<td>150</td>
<td>734</td>
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<tr>
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<td></td>
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<td>Load4</td>
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<td>50</td>
<td>450</td>
<td>64</td>
<td></td>
</tr>
<tr>
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<td>Load5</td>
<td>24</td>
<td>50</td>
<td>450</td>
<td>399</td>
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</tr>
<tr>
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<td></td>
<td>LoadSet2</td>
<td>Load6</td>
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<td>50</td>
<td>450</td>
<td>734</td>
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<td>Potentiometers</td>
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<td>South Array</td>
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<td>30</td>
<td></td>
<td>PressSet2</td>
<td>Press4</td>
<td>19</td>
<td>End of Geogrid</td>
<td>750</td>
<td>400</td>
<td></td>
</tr>
</tbody>
</table>

1. Accelerometer measuring the base excitation (attached to the shake-table base)
2. Accelerometer attached to the rigid box
3.4.1 Wall facing displacement
Two arrays of three potentiometers were used to record wall facing displacement. The potentiometers were located at heights 775, 500 and 200mm with each array located 200mm from each side wall. Potentiometers with odd numbers (Disp1, 3, 5) were grouped and coded as the South Array and similarly even-numbered potentiometers (Disp2, 4, 6), the North Array (Perspex Side-wall side). Layout of the potentiometers is previously shown in Figure 3-9.

3.4.2 Accelerations
A total of 11 accelerometers were used in the model setup with 9 accelerometers embedded within the backfill and the other two accelerometers attached to the rigid box and the shake-table. The accelerometers within the backfill were placed at three different heights within the backfill in the reinforced soil block, the backfill interface and unreinforced backfill locations respectively as shown in Figure 3-9. The accelerometers embedded within the backfill were placed along the rigid box centreline to reduce boundary effects from the wall sidewalls. Accelerometers Acc3, Acc6 and Acc9 were used to quantify the far-field response of the GRS wall with a distance of 800mm from the rigid box end wall to reduce boundary effects.

The accelerometer model AS-2GB manufactured by Kyowa were selected in accordance with accelerometers chosen in Jackson (2010). The accelerometer weighs 25g with the dimensions 14 x 14 x 20mm. The accelerometers were mounted on aluminium plates measuring 50mm by 80mm and 3mm thick, with the total height of the accelerometer being 14mm off the plate surface (Figure 3-10). The cable is integrated with the accelerometer perpendicular to the plate surface and extended roughly 15mm above the accelerometer. Further specifications of the accelerometers are listed in Table 3-7.

![Figure 3-10: Kyowa AS-2GA accelerometer mounted on aluminium plate (Jackson, 2010).](image)

Table 3-7: Accelerometer AS-2GB specifications

<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>
3.4.3 Reinforcement Load Cells

In majority of the GRS wall models tested, 3 out of 5 reinforcement layers were instrumented with load cells to enable the determination of reinforcement loads achieved during excitation of the model. Not all reinforcement layers were instrumented with load cells due to the possibility that the load cells might adversely affect the wall behaviour. The load cells used were 50kg-capacity PT4000 S-type tension load cells that were purchased from PT Limited, Auckland. A diagram showing the shape and dimensions of the load cell is presented in Figure 3-11.

For non-instrumented reinforcement layers (R2 and R4 located at backfill heights of 300mm and 600mm respectively), the reinforcement layer is first attached to the FHR wall panel before placement of the FHR panel within the rigid box. The reinforcement layer is attached to the FHR panel by the use of a steel plate clamped to the FHR panel through a series of bolt and nut systems (Figure 3-12(a)).

In previous research, El-Emam and Bathurst (2007) determined a maximum reinforcement connection load of 1.2 kN/m. Comparatively, the wall models proposed in this research would have a lower vertical spacing between each reinforcement layer and a much higher maximum wall top displacement before each wall model was deemed to fail. As a result, each reinforcement layer was instrumented with a series of three load cells (combined capacity of 1.2 kN/m) to enable the accurate determination of reinforcement loads.

Figure 3-11: Dimensions of PT4000 Load Cell used.

Figure 3-12: (a) Attachment of non-instrumented geogrid layers to wall panel (Jackson, 2010); (b) Instrumented Geogrid detail - Load Cell Connection.
To incorporate the load cells within the reinforcement layer, a portion (140mm) of the geogrid reinforcement was removed and replaced with a series of load cells. It was assumed that the three load cells would supplement the pullout resistance and soil-geogrid shearing resistance that would have been provided by the removed geogrid. This was justified by similar results between KL1 which was not instrumented with load cells and KL4 which was instrumented with load cell. The loss of a portion of the reinforcement in each instrumented reinforcement layer was deemed a necessary sacrifice to enable the determination of the reinforcement loads within each reinforcement layer.

These load cells were attached to the geogrid via a nut and bolt system through steel plates which were clamped down on the geogrid. A rod end purchased from the load cell manufacturer was then attached to the load cell and connected to the wall panel connector using another nut and bolt system. The rod end was used to ensure unlimited mobility of the reinforcement along the y-axis as the FHR wall panel would displace vertically during excitation of the wall model. The subsequent wall panel connectors were bolted to the FHR wall panel when each instrumented reinforcement layer was placed during the model construction process. Load cell connection details are further illustrated in Figure 3-12b and Figure 3-13.

3.4.4 Earth Pressure cells

To enable the determination of residual earth pressures and seismic earth pressures generated within the backfill during excitation, the wall models were instrumented with four earth pressure cells (EPCs) within backfill. The pressure transducers used in the research were TML Soil Pressure transducers of capacity 200kPa (Product Code: KDE-200kPA). Before placement and use of the EPCs in the wall models, each EPC was calibrated by the use of a modified tri-axial cell pedestal. This is performed to determine the calibration factor that converts the cell’s electrical output from voltage (in AD counts) to stress (kPa). The earth pressure cells were embedded at 4 locations within the backfill beneath the R1 and R5 reinforcement layers and at the FHR wall panel and the backfill interface locations as shown in Figure 3-9. As we wish to determine the earth pressures acting on the FHR wall panel during excitation of the wall model, the sensing area of the pressure cells faced away from the FHR wall panel, towards the backfill.
3.4.5 High-speed image capture

Three separate regions of interest (ROI) in the backfill are focused on for GeoPIV analysis: (i) Top of the backfill interface encompassing the ends of the top two reinforcement layer, R4 and R5; (ii) Bottom of the backfill interface encompassing R1 to R3 and (iii) at the wall toe.

ROI (i) and (ii) focused on capturing the inclined failure plane development at the ends of the reinforcement layers in the unreinforced backfill. This relates to previous research conducted by Jackson (2010) that showed the inclined shear planes in the unreinforced backfill originated from the ends of the reinforcement and extended upwards to the surface of the backfill. Furthermore, in Jackson (2010) an observed inclined shear plane extending from the wall toe to the R1 reinforcement layer was observed to have formed in the failure excitation level. Therefore, to further understand shear plane progression at this location, the wall toe is focused on in ROI (iii) with the high-speed camera.

Locations of each region of interest for a representative model are shown in Figure 3-14. The dimensions of each ROI were kept constant for all wall models with only the locations of each ROI changed to suit the varying reinforcement layouts.

The three high speed cameras used were Phantom Miro M310 (Resolution= 1280 × 800); MotionPro X3 (Resolution= 1024 × 1280); and MotionPro Y4 (Resolution= 1016 × 794). Each camera was set to a frame rate of 200 frames per second (fps) and an acquisition time of 12 seconds. Further details of ROI dimensions and cameras used are shown in Section 5.3.3. In addition to the high-speed cameras, a still camera was setup to capture global deformation of the models after each excitation level.
3.5 Model Construction

The staged construction procedure for the GRS wall models were based on procedures developed by Jackson (2010) albeit with slight modifications to the reinforcement connections and instrumentation details. Firstly, the FHR wall panel is braced against the rigid box, and then sand is layered and compacted behind it. During the construction process, each soil layer’s thickness and mass were measured to enable the calculation of average soil density. El-Emam and Bathurst (2004) used a similar method and stated that the sequence of soil and facing placement with the facing column braced can be argued as a construction technique that falls between the field case of an incrementally constructed (unbraced) segmental wall and a FHR panel method.

Staged construction of the GRS wall model involves several steps:

a) Connection of reinforcement layers R2 and R4 to the FHR panel
b) Bracing of the FHR wall for construction
c) Layered placement and compaction of a backfill layer
d) Placement of the reinforcement and earth pressure cells
e) Accelerometer placement
f) Incorporation of black marker sand for both vertical and horizontal lines
g) Un-bracing of wall panel (GRS wall model under static conditions)

3.5.1 Initial bracing of the FHR wall panel

The FHR wall panel is identical to that used by Jackson (2010) in his studies and were dimensioned 960mm by 798mm and 5mm thick (Figure 3-15). 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 (Jackson, 2010).

![Figure 3-15: FHR aluminium panel with attached stiffeners and guides for wall bracing: (a) side view; (b) front view (Jackson (2010)).](image-url)
Figure 3-16: Bracing of the FHR wall panel during model construction

Figure 3-16 shows the initial bracing of the FHR wall during construction. The panel was placed squarely within the rigid box and braced by six braces arranged in two rows and three columns. These braces are screwed tightly into mounted guides on the FHR wall panel, resulting in a consistent wall position achieved for all wall models.

After completion of the model backfill, the braces are removed to allow the wall model to achieve equilibrium under static self-weight. At this stage, some movement of the wall occurred resulting in a reduction of lateral earth pressures towards the active earth pressure and the reinforcement becoming partially engaged (mobilised). This initial displacement is dependent primarily on the stiffness of the reinforcement and its connections, the amount of slack in the reinforcement (mitigated by the application of tension) and the relative density of the backfill. Initial displacement of the FHR wall panel was recorded and is presented in Chapter 4.

3.5.2 Construction of Model deposit

Two different methods for the construction of the GRS wall model have been used in previous studies, both reported by Watanabe et al. (2003) and El-Emam and Bathurst (2004) respectively. Watanabe et al. (2003) used air-pluviation to construct their 500mm high wall models in layers. El-Emam and Bathurst (2004) constructed their models by first depositing the sand in 100mm layers and then vibrating the shake-table and box after each layer to achieve the target density. The method used by El-Emam and Bathurst (2004) was chosen by Jackson (2010) as air-pluviation is time intensive and requires the use of robotic-automated equipment. To maintain consistency between the two phases of experimentation, GRS model construction was performed in an identical manner to Jackson (2010).

The backfill was constructed in layers of 75mm thickness leading to a total of 12 sand layers deposited during the model construction process. To achieve the target densities of the backfill, the
mass and volume of sand deposited for each layer had to be determined. Each layer of sand was compacted to its target density via an added weight through a compaction plate and shake-table excitation (Section 3.5.5). Prior to the backfill deposition, plastic tubes (folded sheets) were taped against the acrylic wall vertically and filled with black sand for the construction of vertical black sand lines (Further illustrated in Section 3.5.4).

**3.5.2.1 Measurement of Sand volume**

To determine the volume of sand deposited for each lift, the thickness of each sand layer deposited was measured. This was performed after backfill compaction with the compaction plate still resting on the backfill as the compaction plate provide level points of reference for the backfill surface. Eqn 3-3 was used to determine the thickness of the sand layer deposited. With the thickness of the sand layer determined, we are able to estimate the total volume of the sand deposited as the width and internal length of the backfill area are known.

\[
\text{Thickness of sand layer } i, t_i = \text{Initial depth of tank} - \text{Depth to compactor plate} - \text{compactor plate thickness} - \text{previously measured height of deposit}
\]

**3.5.2.2 Measurement of Sand mass**

Four storage containers each with a sliding door mechanism were previously constructed for sand storage (Jackson, 2010). A 1000kg load cell was attached to the crane and was used to measure mass of the storage container and the contained sand. The mass of each sand layer is determined as follows:

\[
\text{Mass of sand layer} = \text{Initial Mass of storage container} - \text{postdeposition mass of storage container}
\]

Typically the mass of each sand layer was 245kg for a 85-90% deposit and 229kg for a 50% deposit. Sand is deposited evenly over the backfill area to enable easy levelling of the sand layer. As the soil properties of the sand are known, the relative density \(D_r\) can be determined for each sand layer deposited.

**3.5.2.3 Densification Process**

To densify each sand layer, a combination of a compaction plate of 950kg and shake-table excitation is used. The compactor plate acts as a backfill surcharge, applying a uniform vertical stress of approximately 4.8kPa to the backfill surface under static conditions. With the compactor plate in place, some magnitude excitation is applied to the model via the shake-table. Backfill construction was trialled several times and as a result, the frequency of excitation was determined to be the main parameter in the densification process. Therefore, the frequency content of excitation was varied during the model construction process. The frequency of excitation is typically about 11-13Hz for a \(D_r=90\%\) backfill and 8-9Hz for a \(D_r=50\%\) backfill, each with a small displacement amplitude stroke of \(\pm 1\text{mm}\) for a duration of 10 seconds.

Observations during model construction indicated that as more mass is added to the shake-table (as the model nears full completion height); greater accelerations were measured for the same input motion. As the accelerometers used in the experiments that were placed in the backfill had a maximum capacity of 2.0g, the frequency of excitation has to be constantly monitored and in some
cases, altered to ensure that accelerations greater than 2.0g were not experienced in the backfill during densification process.

### 3.5.3 Reinforcement layer installation and placement

For the non-instrumented reinforcement layers (R2 and R4 reinforcement layers), the geogrid reinforcement is first attached to the FHR panel before placement of the FHR wall panel within the rigid box. After construction of the backfill up to the reinforcement layers, the geogrid reinforcement is laid over the backfill and pulled taut while deposition of the sand layer occurs. The application of tension to the reinforcement layer during sand deposition of the next backfill layer is to ensure maximum soil-geogrid interlock is achieved during the construction process. In Figure 3-17, placement of the non-instrumented R2 reinforcement layer is shown with other non-instrumented R4 reinforcement layer wrapped over the FHR panel at the top of the figure. The transparent acrylic window of the rigid box is shown on the left of the figure.

For the instrumented reinforcement layers, the three load cells within the reinforcement layer are attached to the FHR wall panel through a nut and bolt system with the load cells free to move in the vertical plane (along the y-axis). As previously mentioned in Section 3.4.3, attachment of the geogrid to the load cells was achieved through steel plates which clamp down on the geogrid reinforcement through a series of bolt and nut system. Similarly to the non-instrumented reinforcement layers, after the wall panel connector has been bolted to the FHR wall panel, tension is applied to the reinforcement layer during the deposition of the next backfill layer to increase the soil-geogrid interaction of the reinforcement interface.

![Figure 3-17: (a) Placement of the non-instrumented R2 reinforcement layer; (b) Placement of instrumented R5 reinforcement layer.](image-url)
3.5.4 Vertical and Horizontal lines of black marker sand

To enable visualisation of the global deformation of the backfill, vertical and horizontal lines of black marker sand were used in the model setup. The use of coloured sand lines for this purpose is common in literature with Jackson (2010), Watanabe et al. (2003) and El-Emam & Bathurst (2004) all cataloguing their use in their papers.

Black sand was achieved by dying the Albany sand with black ink and subsequently oven-drying the dyed sand. After being oven-dried, the dyed sand typically formed semi-solid blocks which were broken down into the individual sand particles using a mortar and pestle. After going through the mortar and pestle, the dyed sand was then sieved to ensure the same grain size as the original Albany sand backfill was achieved.

Vertical black sand columns were constructed at every 150mm intervals between each layer of reinforcement. To construct the vertical columns of black sand, folded plastic sheets of approximately 300mm in length and 8mm width were taped to the inside of the transparent acrylic wall as shown in the series of images shown in Figure 3-18. The folded sheets were sourced from plastic sheets which were typically used as protective sheets for bound documents. The black sand was then funneled into the void between the folded sheets and the transparent side wall.

![Figure 3-18: (a) Removal of the tape after compaction of the respective sand layer; (b) Plastic sheet shifted upwards with a portion still embedded within the backfill; (c) The plastic sheet filled with dyed sand and taped to the acrylic sidewall awaiting construction of the next two sand layers.](image-url)
The tape for the folded sheets was looped back up to the top of the tube to enable easy removal of the tape. After removal of the tape, the plastic sheets are held up against the wall by sand pressure alone. To construct the vertical black sand columns for the next interval, the plastic sheets are shifted upwards (~200mm), with about 50mm of the plastic sheet still embedded within the sand. The process of taping the plastic sheets to the side of the wall and funnelling of the black sand is then repeated.

The process of removing the tape for the vertical black sand lines tended to leave the sand in an uneven state at the box sidewalls. As a result, construction of the horizontal black sand lines and the placement of the geogrid were performed after the construction of the black sand columns. At the transparent sidewall, the sand was scraped flat and a slight indentation was made for the funnelling of the black sand for the horizontal sand lines. The construction of the horizontal black sand lines were performed after the placement of the geogrid to ensure the geogrid reinforcement was located within the horizontal black sand line (Figure 3-19).

Jackson (2010) noted that the image texture for GeoPIV analysis was increased with the additional of the horizontal and vertical lines of black sand. However, further in-depth analysis of Jackson (2010)’s work show that due to the different colouration of the black sand lines, the GeoPIV software detected the black sand lines as zones of small dilation although no strain development had formed, indicating some inaccuracies in the GeoPIV strain analyses. As a result, for this research, the black sand lines were not continuously constructed through the high-speed camera window locations as shown in Figure 3-14 to prevent misinterpretation by the GeoPIV analysis.

Figure 3-19: Deposition of black dyed sand for horizontal sand line construction after placement of the geogrid.
3.5.5 Wall model deposit construction
In this section, the initial progression of model deposit construction is shown as a series of images. The images show progression from the initial placement of the L1 sand layer up till the placement of geogrid on the compacted and levelled L4 sand layer at height 300mm.

(a) Empty box with vertical black sand lines for the first two sand layers using thin vertical tubes taped to the acrylic window.

(b) Deposition of first sand layer, L1

(c) Loose sand after raking and levelling, pre-compaction.

(d) Placement of the compactor plate and subsequent vibration of the shake-table at 8-11Hz for 10s for sand compaction

(e) Post-compaction and removal of the compactor plate
(f) Deposition of sand for the next sand layer, L2 on top of the compacted L1 layer.

(g) Compaction of the L2 sand layer after levelling of the loose L2 sand layer.

(h) Placement of the instrumented R1 reinforcement layer and the construction of the horizontal black sand lines along the R1 reinforcement layer. Extension of the vertical black sand lines for the next two sand layers was also performed at this stage of construction.

(i) Sand layer L3 post-compaction. The accelerometers are placed on top of the compacted fill.

(j) Sand layer L4 post-compaction with the placement of the non-instrumented R2 reinforcement.

Figure 3-20: Model construction of the first 4 sand layers presented as a series of images (a) to (j).
3.5.6 Surcharging

To achieve the surcharge layout proposed in Section 3.2.5, staged construction of the surcharge was performed. This is shown in the series of images below (Figure 3-21). During the placement of the surcharge, the wall braces were kept in place and only removed after full completion of the surcharge. Care was taken to ensure the surcharge bags did not touch the rigid box sidewalls to prevent dynamic interference of the rigid box on the surcharge. Note, FHR wall displacement measurements were taken before and after wall brace removal to record the static displacement of the FHR wall under surcharging.

Figure 3-21: (a) Backfill surface before placement of the surcharge; (b) Placement of the foam material on the surface of the backfill; (c) Placement of the steel plates on the foam material; (d) Steel plates on the foam material; (e) Completed layout of steel plates and surcharge bags; (f) Plan view of the surcharge (far-field end).
3.6 Model Deposit density

As mentioned in the previous sections, model deposit construction involved deposition of the sand in layers of 75mm and a compaction process that involved the use of a compactor plate of 950kg combined with shake-table excitation at 8-11Hz for 10 seconds. Due to the densification of the backfill, the compactor plate typically drops approximately 5-7mm from its pre-compaction position. After compaction, the depth of the compactor plate with respect to the top of the rigid box is measured at 300mm intervals. By using the known thickness of the compactor plate and the determined depth from the top of the rigid box to the base of the model, these measurements enable the determination of an average layer thickness due to the rigidity of the compactor plate.

Table 3-8: Determined wall model backfill densities

<table>
<thead>
<tr>
<th>Wall Model</th>
<th>Total Sand Mass (kg)</th>
<th>Backfill Height (mm)</th>
<th>Unit Weight (kN/m³)</th>
<th>Void ratio, e</th>
<th>Relative Density, Dr (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>KL1</td>
<td>2976</td>
<td>906.5</td>
<td>16.8</td>
<td>0.547</td>
<td>88.3</td>
</tr>
<tr>
<td>KL2</td>
<td>2916</td>
<td>899.5</td>
<td>16.6</td>
<td>0.567</td>
<td>82.2</td>
</tr>
<tr>
<td>KL3</td>
<td>2732</td>
<td>902.0</td>
<td>15.5</td>
<td>0.677</td>
<td>47.7</td>
</tr>
<tr>
<td>KL4</td>
<td>2931</td>
<td>899.4</td>
<td>16.7</td>
<td>0.559</td>
<td>84.7</td>
</tr>
<tr>
<td>KL5</td>
<td>2741</td>
<td>900.4</td>
<td>15.6</td>
<td>0.669</td>
<td>50.4</td>
</tr>
<tr>
<td>KL6</td>
<td>2741</td>
<td>901.7</td>
<td>15.6</td>
<td>0.671</td>
<td>49.6</td>
</tr>
<tr>
<td>KL7</td>
<td>2743</td>
<td>901.7</td>
<td>15.6</td>
<td>0.670</td>
<td>50.0</td>
</tr>
<tr>
<td>KL8</td>
<td>2741</td>
<td>901.6</td>
<td>15.6</td>
<td>0.671</td>
<td>49.7</td>
</tr>
<tr>
<td>KL9</td>
<td>2746</td>
<td>900.0</td>
<td>15.6</td>
<td>0.665</td>
<td>51.6</td>
</tr>
<tr>
<td>KL10</td>
<td>2741</td>
<td>901.4</td>
<td>15.6</td>
<td>0.671</td>
<td>49.8</td>
</tr>
<tr>
<td>KL11</td>
<td>2742</td>
<td>901.0</td>
<td>15.6</td>
<td>0.669</td>
<td>50.2</td>
</tr>
<tr>
<td>KL12</td>
<td>2742</td>
<td>901.4</td>
<td>15.6</td>
<td>0.670</td>
<td>50.0</td>
</tr>
</tbody>
</table>

Table 3-8 shows the average relative densities achieved for each of the wall models constructed. The similarity between the total sand mass within each wall model and backfill height achieved indicate consistent model construction processes and repeatability of target backfill density.

To achieve a high backfill density of 90%, a shake-table frequency of 13Hz was needed. However, when the model was excited at frequencies greater than 11Hz, high accelerations were recorded in the backfill of greater than 2.5g, which was beyond the capacity of the accelerometers embedded within the backfill. Therefore, to limit the risk of damaging the accelerometers, shake-table excitation was limited to a maximum frequency of 11Hz. Although a backfill density of about 80-85% was only achieved for the KL1, KL2 and KL4 wall models, these wall models are still classed as high density wall models due to the relative low density of 50% achieved in the lower density wall models.
3.7 Summary

For the 1-g shake-table tests on reduced-scaled GRS wall models proposed for this study, the GRS wall model setup previously used by Jackson (2010) was used. A change of the wall seal material was made due to failure of the wall seal and maintained for the wall models KL4 to KL9. The reinforcement selected was a Stratagrid Microgrid and Albany sand was selected as the backfill soil due to its ready availability and known soil properties. Difficulties in modelling the geogrid reinforcement are noted with the Microgrid being under-scaled in terms of its stiffness and the complex soil-geogrid relationship not accounted for in current scaling laws. The theoretical effect of a 3kPa backfill surcharge is also presented in terms of increased vertical and horizontal stresses.

Model excitation involved the use of the University of Canterbury shake-table and was identical to Jackson (2010) to maintain consistency. A stepped amplitude (at 0.1g increments) sinusoidal function of 5Hz predominant frequency and 10 seconds duration was used as the base excitation. The excitation frequency was determined to be lower than the natural frequencies of the testing system of 13.7 Hz and 14.8 Hz for the 90% and 50% relative density wall models.

A maximum of 30 instruments were used in each wall model. This included 11 accelerometers, 9 load cells, 6 potentiometers and 4 earth pressure cells. Furthermore, high speed image capture of selected regions of interest within the backfill was achieved to recorded backfill deformation for GeoPIV analysis. The GRS model construction was performed in a systematic and controlled manner where the model backfill was constructed in a series of lifts of 75mm. Densification of each layer to the target density was achieved by using the vibration method; via a compaction plate and shake-table excitation. Details of load cell installation on the reinforcement layers and methodology of surcharge placement and layout were also presented in this chapter. Lastly, model deposit densities achieved for each wall model were summarised.

Test results are presented and discussed in Chapter 4 and 5.

3.8 References


Chapter 4.0  EXPERIMENTAL RESULTS

4.1 Introduction
A series of twelve reduced-scale GRS wall model shake-table tests were conducted at the University of Canterbury. The tests investigated the influence of reinforcement length ratio (L/H), reinforcement layout, soil density and surcharging on the seismic performance of the GRS walls. The first three tests were repeated in subsequent tests due to some errors and differences in the testing procedure with results from tests KL4 to KL12 are the main focus of this chapter. A testing summary is presented in Section 4.1.

Section 4.2 investigates the reliability of the experimental results collected throughout the series of tests. Potential installation effects on data collected and initial problems with testing protocol are discussed in this section. Section 4.3 concentrates on the typical results of one of the tests performed, KL5. This test was selected because the results from the test were most representative of GRS behaviour under seismic loading of all wall models. Discussions of typical GRS wall behaviour and responses are made in this section.

In the subsequent Section 4.4, results from all tests are used to analyse the effect of different the GRS wall parameters (the reinforcement length ratio, backfill density and surcharging) on the behaviour and response of the GRS wall such as acceleration amplification, modes of failure etc. Lastly, Section 4.5 summarises the findings of the chapter.

4.1 Testing Summary
A series of twelve shake-table tests were performed on reduced-scaled GRS wall models. For each test, the reinforcement length ratio and layout, soil density and presence of surcharge were varied. Some tests were repeated due to issues with the controllability of the shake-table amplitude as well as model construction differences. An identical model excitation regime to Jackson (2010) was selected to ensure consistency in testing procedures between the two phases of experimental tests. Each wall model was tested with a sinusoidal input motion with a predominant frequency of 5Hz and duration of 10s. A first excitation level of 0.1g acceleration amplitude was chosen and each excitation level was increased by 0.1g increments until failure of the wall.

In the first three tests, the wall models were not instrumented with reinforcement load cells. This was due to the time constraints present in Stage I testing and the uncertainty of the effect the load cells will have on the behaviour of the GRS wall. Furthermore, there were wall seal issues present in KL2 and KL3 where failure of the wall seal occurred in KL2 and a high friction wall seal was used in KL3. The wall seal issue was resolved in the subsequent wall models (KL4 to KL12) and is touched on in Section 4.2.1.2. Additionally, shake-table problems in KL1 resulted in inconsistent dynamic input imparted to the wall model at the 0.1g and 0.2g excitation level. As a result, excitation of wall models KL2 and KL3 were commenced at the 0.3g excitation level. This issue is further elaborated on in Section 4.2.1.1. Wall model KL4 was a repeat of KL1 with identical reinforcement lengths but with a complete set of instruments. Both wall models failed at the 0.6g excitation level indicating consistency and suitability of the new wall seal and the presence of load cells within the wall model. No experimental issues were found for KL5.
Table 4-1: Test summary of parameters varied and problems during each test

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Reinforcement Layout, L/H</th>
<th>Density %</th>
<th>Surcharge (kPa)</th>
<th>Acceleration at failure (g)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Stage I</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>KL1</td>
<td>0.75</td>
<td>89</td>
<td>0</td>
<td>0.6</td>
<td>No Load cells and earth pressure cells, Inconsistent shake-table cycles, Wall Seal A</td>
</tr>
<tr>
<td>KL2</td>
<td>R5 @ 0.9, R4 @ 0.75 Rest @ 0.6</td>
<td>85</td>
<td>0.5</td>
<td>No Load cells, Inconsistent shake-table cycles, Wall Seal A (seal issues)</td>
<td></td>
</tr>
<tr>
<td>KL3</td>
<td>0.75</td>
<td>50</td>
<td>0.5</td>
<td>No Load cells, Inconsistent shake-table cycles, Wall Seal B</td>
<td></td>
</tr>
<tr>
<td><strong>Stage II</strong></td>
<td>-</td>
<td></td>
<td></td>
<td></td>
<td>Wall Seal C used in all Stage II tests</td>
</tr>
<tr>
<td>KL4</td>
<td>0.75</td>
<td>85</td>
<td>0.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>KL5</td>
<td>0.75</td>
<td>50</td>
<td>0.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>KL6</td>
<td>0.9</td>
<td>50</td>
<td>0.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>KL7</td>
<td>0.9</td>
<td>50</td>
<td>0.6</td>
<td>Failure of 1 of 4 Earth Pressure cells</td>
<td></td>
</tr>
<tr>
<td>KL8</td>
<td>Top two layers @ 0.9 Rest @ 0.75</td>
<td>50</td>
<td>0.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>KL9</td>
<td>Top two layers @ 0.9 Rest @ 0.75</td>
<td>50 3</td>
<td>0.5</td>
<td>Large initial rotation of wall facing under static conditions</td>
<td></td>
</tr>
<tr>
<td>KL10</td>
<td>0.9</td>
<td>50</td>
<td>3</td>
<td>0.6</td>
<td></td>
</tr>
<tr>
<td>KL11</td>
<td>0.75</td>
<td>50</td>
<td>3</td>
<td>0.5</td>
<td></td>
</tr>
<tr>
<td>KL12</td>
<td>Top two layers @ 0.9 Rest @ 0.75</td>
<td>50 3</td>
<td>0.5</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

In KL6, due to human error, the data from the data-logger for the 0.5g excitation level could not be retrieved, resulting in no time histories at the 0.5g excitation level for KL6. Residual values of the instruments for this testing stage could still be determined from the difference between the residual values at the 0.4g level and the initial values recorded at the 0.6g level. Furthermore, during the failure excitation level of KL6 testing, the earth pressure cells (Press2) located at the bottom of the backfill interface failed due to overextension of its cable, possibly due to entanglement with the load cell cables. Therefore, for subsequent wall models KL7 to KL12, only three earth pressure cells were used instead of four.

No experimental issues were found for KL7 and KL8. A 3kPa backfill surcharged was applied to the wall models KL9 to KL12. However, in KL9, the wall model displayed a large initial displacement under static conditions of 2.34mm wall top displacement compared to similar surcharged models of different reinforcement length, 0.87mm for KL10 and 1.11mm for KL11. It is thought that this may be due to excessive bulging of the Perspex window due to the surcharge which was mitigated in the subsequent surcharged tests by leaving some of the Perspex braces in place during model excitation. KL12 was a repeat of KL9 as the behaviour of the KL9 wall during excitation could have been affected by the large initial displacement under static conditions. Excitation of the KL12 wall model was conducted with the Perspex braces in places as for KL10 and KL11.
As a result of the issues regarding experimental procedure and the reduced instrumentation of the wall models in Stage I, their results were deemed unreliable and hence are not used for analysis. Results from the KL9 wall model are also used sparingly in analysis as its large initial displacement under static conditions may have affected the performance of the GRS wall during excitation.

Time histories of the backfill accelerations, wall displacements, geogrid reinforcement load and earth pressures for all wall models are presented in Appendix B at the end of this thesis for reference. Due to the high frequency spikes in each of the time histories (“noise”), the time histories are filtered (identical filter as stated in Section 4.2.1.1) for clearer presentation of the time histories.

4.2 Reliability of experimental results
In this section, the reliability of test results are discussed in relation to issues encountered during the experimental procedure as well as possible errors in the experimental setup and instrumentation and any installation effects on parameters of interest.

4.2.1 Issues during experimental procedure
In this section, issues during experimental procedure are discussed. This includes the consistency of dynamic input imparted to the wall models in Stage I testing and wall seal issues in KL2 and KL3.

4.2.1.1 Consistency of dynamic input for tests
To compare the consistency of dynamic input for all tests, the base input accelerations are analysed in terms of acceleration time histories and strong motion intensity measures. An 8th order Butterworth filter with a lowpass frequency of 10Hz was applied to the accelerometer data in accordance with previous analyses by Jackson (2010).

4.2.1.1.1 Acceleration Time Histories
In the first stage of testing (for KL1, KL2 and KL3), there were issues with the controllability of the shake-table amplitude at lower excitation levels (below 0.3g). During testing, it was found that the actual displacement of the shake-table during excitation was not consistent with the displacement inputted into the control program. For KL1, the acceleration data recorded for the lower excitation levels (0.1g to 0.2g) were much greater than the target values, with ‘pulses’ of acceleration recorded instead of the consistent cycles expected (Figure 4-1 & Figure 4-2). Only from the 0.3g excitation level onwards, the filtered acceleration time histories showed consistent cycles of base input acceleration achieved (Figure 4-3). Therefore to ensure consistency in model excitation, excitation of the KL2 and KL3 models were commenced from the 0.3g excitation level onwards (Figure 4-4).

In the second stage of testing (KL4 – KL12), the issues with the shake-table were resolved after re-tuning and re-calibration of the controls. Base input accelerations applied to the models were in consistent cycles and close to targeted acceleration values. Figure 4-5 shows the base input accelerations recorded for KL4 at the 0.1g excitation level.

Figure 4-1: Base Input Acceleration for KL1 at the 0.1g excitation level (Filtered).
4.2.1.1.2 Intensity parameters used to quantify excitation

Besides comparing acceleration time histories between tests, strong motion intensity parameters are also used to determine the consistency of dynamic input between the models. These are first identified in this section.

The most commonly used intensity parameter to quantify dynamic excitation is the peak ground acceleration (PGA) or peak horizontal acceleration. However, peak accelerations typically cause insignificant damage to structures because they occur at very high frequencies. Furthermore, PGA does not provide information on the frequency content and duration of motion and must be supplemented with additional information to characterize a ground motion accurately (Kramer, 1996).

One intensity measure that can be used is the RMS acceleration, where the RMS acceleration parameter includes the effects of the amplitude and frequency content of the acceleration record, defined in Eqn (4-1). However, it is noted that the RMS acceleration parameter can be sensitive to the method used to define strong motion duration (Kramer, 1996).
\[ a_{rms} = \sqrt{\frac{1}{T_d} \int_{0}^{T_d} [a(t)]^2 \, dt} = \sqrt{\lambda_0} \]  

(4-1)

Where: 
- \( a_{rms} \) is the RMS acceleration parameter; 
- \( T_d \) is the duration of strong motion; 
- \( \lambda_0 \) is the average intensity (mean-squared acceleration); 
- \( a(t) \) is the acceleration time history.

The Arias intensity parameter which is closely related to the RMS acceleration can also be used as an intensity measure for excitation (Arias, 1970) and is defined as:

\[ I_a = \frac{\pi}{2g} \int_{0}^{\infty} [a(t)]^2 \, dt \]  

(4-2)

Where: 
- \( I_a \) is the Arias intensity parameter; 
- \( g \) is the gravitational acceleration constant; 
- \( a(t) \) is the acceleration time history.

As the Arias parameter is obtained by integration over the entire duration of motion rather than over the duration of strong motion, its value is independent of the method used to define the strong motion duration of an acceleration record (Kramer, 1996).

Both RMS acceleration and Arias intensity parameters are suitable to quantify the sinusoidal excitation imparted to the wall models, as the strong motion duration is typically the entire duration of motion in a sinusoidal function.

In addition to this, the sustained maximum acceleration (SMA) parameter can also be used to quantify the dynamic excitation of the model. The SMA parameter is defined by Nuttli (1979) as the third (or fifth) highest (absolute) value of acceleration in the time history.

### 4.2.1.1.3 Comparisons using intensity parameters

To compare the consistency of excitation between the tests, the tests from the first stage of excitation (KL1, KL2 and KL3) are compared with KL4 and KL5 which had consistent cycles observed from acceleration time histories at all stages of excitation.

Using PGA and SMA as a basis for comparison (Figure 4-6), a noted difference is observed for the KL1 model compared to the other wall models with larger PGAs and SMAs recorded in the 0.1g and 0.2g excitation step as compared to KL4 and KL5. The PGA experienced in the 0.2g excitation level in KL1 is 0.246g compared to a maximum of 0.23g experienced in tests KL4 to KL12. Similar differences in SMA are observed. However, it is noted that PGA is not a good measure of strong ground motion intensity as peak accelerations usually occur at high-frequencies, thus having minimal effect on the performance of the structure.
No significant difference was observed between KL1 and KL4 when comparing Arias intensity and RMS acceleration with the largest difference of 0.01g determined in the final excitation level. This indicates that despite the non-consistent cycles of the base input acceleration observed in the 0.1g and 0.2g acceleration time histories for KL1, dynamic excitation imparted to the KL1 model was relatively consistent with the other wall models. In other words, actual dynamic input of the shake-table to the wall model in KL1 was fairly consistent with the other tests that were conducted with the shake-table recalibrated, i.e. KL4-KL12.

Testing of the KL2 and KL3 models were commenced from the 0.3g excitation level due to the concern that the dynamic input at the lower excitation levels was inconsistent (based on KL1 time histories). However, in light of the current analysis, this assumption was inaccurate as the RMS acceleration comparisons show good consistency between KL1 and KL4 in all excitation levels (Figure 4-7). The problems with the shake-table in the first three tests contributed to more “noise” in the acceleration data in the form of higher peak accelerations but this did little to change the dynamic input level due to the high frequency at which they occur.

Therefore, we can conclude that there is a strong consistency of the dynamic input across the tests.

Figure 4-6: Comparison of (a) PGA and (b) SMA recorded for base input accelerations between KL1, KL2, KL3, KL4 and KL5.
4.2.1.2 Failure of wall seal in initial tests

The original wall seal used by Jackson (2010) was based on that used by Watanabe et al. (2003) and consisted of Teflon tape and foam. This wall seal (Wall Seal A) was used for KL1 and KL2. It is noted that in this study, Wall Seal A resulted in sand leakage during model construction and testing. This sand became trapped between the Teflon sheet and the side wall and collected at the bottom of the wall seal.

However, after construction of KL2, failure of the wall seal at the non-Perspex sidewall occurred, leading to leakage of sand. Sand leakage was stopped by pressing the wall seal in place with a metal rod. The failure of the wall seal in KL2 led to a thicker and wider strip of foam being used for the seal in KL3 (Wall Seal B). However, the new wall seal was too tight and resulted in a much reduced wall displacement (the wall was unable to rotate fully) compared to what was expected for KL3. As a result, new material was used for the wall seal in subsequent tests, KL4 to KL12 (Wall Seal C). The new seal consisted of a strip that was made from ultrahigh-molecular weight polyethylene which was stiffer than the previously used Teflon strip. However, its coefficient of friction of 0.10 to 0.22 is comparable to Teflon’s frictional coefficient of 0.40 to 0.25.

To test the wall seal, the force required to overturn the wall without any backfill was determined. Using a digital spring balance, we determined that a force of 5 N applied to the top of the wall would result in overturning and failure of the wall. This implies that the resistance of the wall seal is minimal and plays a minor part in the overturning resistance of the wall.

During testing, Wall seal C did not result in any sand leakage. When wall seal A was used, sand pooled at the base of the wall panel and was stuck between the Teflon sheet and the sidewall. This would have resulted in an increased frictional resistance of the wall seal at the base of the wall panel. Therefore, Wall Seal C is a more suitable seal to model a frictionless seal.

Furthermore, note that KL4 failed at the same excitation level as KL1 with both wall models having identical reinforcement length. This indicates the change in wall seal and the addition of load cells to the reinforcement layer in KL4 did not have a significant effect on the performance of the GRS wall. Thus, indicating the suitability of the experimental setup.
4.2.2 Wall displacement symmetry
One of the assumptions made during testing is that the FHR wall panel remains rigid and maintains its linear profile during all stages of testing. Furthermore, it was assumed that displacement of the wall panel is symmetrical about the wall centreline. To determine the validity of these assumptions, wall displacement profiles of a representative test (KL5_LH075_DR50) are analysed in the following section.

At all stages of excitation, a difference in displacements recorded by the North and South arrays of potentiometers is observed with the North array recording greater displacement. The maximum difference in displacement recorded is about 3mm (or approximately 2% difference) after the failure excitation step (Figure 4-8). The non-symmetrical displacement of the wall panel is likely due to the slight bulging of the Perspex sidewall (North) which would have reduced the frictional resistance of the wall seal thus influencing the wall displacement. Slight bulging of the Perspex sidewall was due to the removal of the Perspex sidewall brace located at the wall panel location. This was done to enable image-capture of the wall toe for GeoPIV analysis during testing. As the difference in displacement recorded is relatively small of 2% total wall displacement, it is deemed to have little influence on the general wall behaviour during excitation.

The FHR wall panel generally does maintain a linear profile during testing. A slight non-linearity of the wall is observed in Figure 4-8(a), under static conditions but is not observed in subsequent stages of testing. It is likely that the wall profile is initially non-linear due to slight deformations experienced by the facing panel under static conditions but reverts back to a linear profile after excitation. The deviation of the wall profile from a linear profile is very small (0.2mm for the North displacement values recorded in KL5). Therefore, it can be assumed the non-linearity of the wall profile only observed when the wall is under static conditions does not have a significant influence on the general wall behaviour.

![Figure 4-8: Cumulative Displacement recorded by the North and South arrays of potentiometers: (a) After wall strut removal; (b) After 0.1g excitation; (c) At failure.](image-url)
4.2.3 Installation effects on reinforcement loads

As discussed in Section 3.4.3, the load cells were first attached to the geogrid through rigid steel plates before placing them in the backfill and bolting them to the FHR wall panel. During installation of the load cells, the North and South load cells were found to be slightly further away (1-2mm) from the FHR wall panel compared to the centreline load cells. This indicated that there was a slight inward bow along the centreline of the FHR wall facing. Therefore upon the installation of the reinforcement layer (by bolting them to the facing), the North and South load cells were subjected to tension force and the Centreline load cell was subjected to a compression force as observed in Figure 4-9.

As a result of the bow in the wall, the process of bolting the load cells (with the geogrid attached) to the FHR wall panel develops non-uniform loads across the three load cells in the reinforcement layer. Initially, internal forces generated within the reinforcement layer immediately after installation of each reinforcement layer are in relative equilibrium. This is evidenced by the relatively low sum of internal forces within each reinforcement layer shown in Table 4-1. Note that in the table, tension is positive and compression is negative and that the sum of the loads is near zero – refer to the last two columns.

Table 4-1: Reinforcement Load (N) recorded immediately after load cell installation – KL12

<table>
<thead>
<tr>
<th>Elevation</th>
<th>North-Side</th>
<th>Centreline</th>
<th>South-Side</th>
<th>Sum of Internal Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td>Units</td>
<td>mm</td>
<td>N</td>
<td>N</td>
<td>N</td>
</tr>
<tr>
<td>R1 Layer</td>
<td>150</td>
<td>111</td>
<td>-258</td>
<td>108</td>
</tr>
<tr>
<td>R3 Layer</td>
<td>450</td>
<td>73</td>
<td>-147</td>
<td>73</td>
</tr>
<tr>
<td>R5 Layer</td>
<td>750</td>
<td>-32</td>
<td>66</td>
<td>-32</td>
</tr>
</tbody>
</table>

* Percentage of maximum internal force experienced within reinforcement layer

Figure 4-9: Reinforcement loads recorded in KL7 prior to wall strut removal.
Upon further construction of the model, this initial force equilibrium does not remain. Non-uniform reinforcement load development due to further sand layers constructed on top of the reinforcement layer is observed. For the North and South load cells, a decrease in reinforcement load is recorded (increased compression) and the Centreline load cell develops an increase in reinforcement load (increased tension), as shown in Table 4-2. As a result, prior to excitation of the wall models, a net compressive force is observed in the instrumented reinforcement layers for most of the wall models. This net compressive force is developed purely due to the interaction between the wall panel and the steel strip to which geogrid is attached as no deformation of the FHR wall panel or the backfill has occurred.

To determine the effect that the displacement of the wall facing has on the reinforcement loads developed, we wish to isolate the initial reinforcement loads developed due to the internal mechanisms within the reinforcement layer. To do so, reinforcement loads that are developed prior to removal of the FHR wall struts are removed from the subsequent load cell readings.

### Table 4-2: Reinforcement Load recorded after completion of model construction - KL12

<table>
<thead>
<tr>
<th>Elevation</th>
<th>North-Side</th>
<th>Centreline</th>
<th>South-Side</th>
<th>Sum of Internal Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit</td>
<td>mm</td>
<td>N</td>
<td>N</td>
<td>N</td>
</tr>
<tr>
<td>R1 Layer</td>
<td>150</td>
<td>44</td>
<td>-270</td>
<td>35</td>
</tr>
<tr>
<td>R3 Layer</td>
<td>450</td>
<td>19</td>
<td>-52</td>
<td>-3</td>
</tr>
<tr>
<td>R5 Layer</td>
<td>750</td>
<td>-57</td>
<td>97</td>
<td>-56</td>
</tr>
</tbody>
</table>

* Percentage of maximum internal force experienced within reinforcement layer

4.2.3.1 **Effect on incremental load development under static load**

For all wall models, an increase in tension load is experienced in all load cells due to the mobilization of the geogrid as the FHR wall panel rotates slightly under self-weight upon removal of the FHR wall struts.

For wall models KL5 & KL9 in particular, the R5 centreline load cell exhibits a significantly greater reinforcement load development compared to the North and South load cells. This is because both models experience larger than normal initial displacement of the FHR wall panel (1.6mm & 2mm at the wall top for tests KL5 and KL9, respectively), resulting in the mobilization of the geogrid and subsequent development of tension load (positive reinforcement load) in the top reinforcement layer.

However for most of the wall models tested, under static load, the Centreline load cells experience lower reinforcement load increments (i.e. less tension) than the North and South load cells. This indicates that upon removal of the wall struts, the Centreline load cells experience an increase in compressive force along with the expected tension force increment. The increase of compression force is likely to be due to the interaction between the rigid steel plate and the FHR wall panel. Prior to installation, the FHR wall panel is initially slightly bowed. Upon installation, the FHR wall struts then act to reduce this original bowed state. As the wall struts are removed prior to testing, the FHR wall panel tries to return to its original bowed state. However, it is unable to do so due to the rigidity of the steel plates that were bolted to the wall facing. This results in an increase in tension force in the North and South load cells and an increase in compressive force for the centreline load cell.

Therefore, the Centreline incremental reinforcement load cell experiences a combination of both (i) compressive forces due to interaction between the steel plate and FHR wall panel (\(-ve\)) and (ii)
tension forces due to outward movement of FHR panel (+ve). Comparatively, the North and South load cells experience (i) tension forces due to FHR wall panel – steel plate interaction (+ve) and (ii) tension forces due to outward movement of FHR panel (+ve).

4.2.4 Issues with Earth Pressure Data Recorded
In the earth pressure time histories recorded, the max earth pressure recorded is less than 20 kPa. Relative to the capacity of the earth pressure cell which is 200kPa, the earth pressures measured in these tests are too small relative to size of the pressure cells (< 10% of capacity). This lead to large inaccuracies of measurement due to the coarseness of the pressure cell relative to the earth pressures experienced. This is further evidenced by the fluctuations of the pressure readings even though under a constant pressure which was observed during calibration process and during the determination of \( P_{atm} \) pressure. -- Coarseness of measuring instrument relative to magnitude of earth pressures to be recorded.

Furthermore, during model construction, the earth pressure cells were embedded into the backfill after compaction of the respective sand layer with the backfill re-compacted around the embedded pressure cell after placement. Re-compaction was performed using a scraper, a relatively crude method of re-compaction as there was no proven, repeatable method to re-compact the sand. It is assumed that the crude method of compaction would be supplemented by re-compaction of the backfill due to further excitation of the model for subsequent lifts. The disturbance of the sand during placement of the pressure cells could have led to inconsistent backfill densities around the pressure cell, leading to different initial cell readings. -- Possible non-homogeneous backfill around the pressure cells

4.2.5 Differences between identical models
For the identical KL6 and KL7 tests, significant differences between the wall displacement component ratios are observed in the 0.1g to 0.3g excitation levels as shown in Figure 4-10. In Figure 4-10, the sliding displacement ratio, defined as the ratio between the horizontal displacement of the wall toe and the total wall horizontal displacement of the FHR wall panel top, for both wall models are plotted against the excitation level imparted to the wall models. The difference in sliding displacement ratios between the two models decrease with increasing excitation levels from a maximum difference of 0.1 in the 0.1g excitation level to identical ratios achieved in the 0.5g and 0.6g excitation level.

However, from displacement-acceleration curves, both models generally display similar developments of sliding and rotational displacement components in the 0.1g to 0.3 excitation levels (Figure 4-11). The significant difference observed in the sliding displacement ratio plots for the identical models in Figure 4-10 is likely due to the calculations undertaken to obtain each displacement component ratio. In the initial excitation levels, small differences in displacement components are magnified due to the relatively small wall top displacements. Similarly, at higher excitation levels, these small differences between the wall models are not shown in the displacement component ratio plots. This is shown in Figure 4-10 where the differences in displacement component ratios between the two models achieves a maximum of 0.1 in the 0.1g excitation level and becomes non-existent in the 0.5g and 0.6g excitation level.
Figure 4-10: Sliding displacement component ratio determined for KL6 and KL7.

Figure 4-11: Cumulative (a) sliding and (b) rotational components of wall displacement for KL6 and KL7.
It is noted that from incremental displacement data, the KL6 model showed incremental decrease of sliding displacement in the 0.2g and 0.3g excitation levels (of 0.06mm) compared to an incremental increase exhibited in the KL7 model (of 0.03mm). This reduction of sliding displacement is also observed in Jackson (2010) and is likely the result of the wall toe being ‘stuck’ in the two excitation levels. However, the magnitude of the incremental decrease in sliding displacement is very small and does not have an effect on the overall wall behaviour as sliding becomes more significant at higher excitation levels.

Therefore, by using the ‘displacement component ratio’ method of analysis, key observations of wall behaviour at higher excitation levels can be made but observations made for the initial excitation levels will have to be treated with care.

4.3 Results of a typical case
This section concentrates on the typical results of the wall model KL5 with a uniform reinforcement length ratio (L/H) of 0.75 and a backfill relative density of 50%, highlighting representative features of all tests performed. This will help provide a framework for analyses of different wall parameters and their effect on the performance of the GRS wall which are discussed in the later sections.

4.3.1 General Deformation of GRS wall
To provide an overview of the deformation of the GRS wall under excitation, the gradual displacement of the wall panel at the end of each excitation level (after each stage of shaking) is presented in Figure 4-12. It is observed that sliding displacement of the wall panel only becomes significant in the last two stages of excitation (0.4g and 0.5g) with rotational displacement the predominant component of displacement in all excitation levels. At the 0.1g excitation level, maximum wall displacement (predominantly rotation) of about 5mm at the top of the wall is observed or approximately 0.5% of the wall height. This increases to 20mm and 185mm at the end of the 0.3g and 0.5g excitation levels which is 2% and 20% of the wall height respectively.

Figure 4-12: Horizontal Displacement (mm) of wall panel at the end of each excitation level.
The magnitude of backfill deformation with each excitation level is shown through a series of global images at the end of each excitation level (Figure 4-13).
In terms of backfill deformation, very minimal deformation is observed at the end of the 0.1g excitation level with the slight rotation of the wall panel of 0.5% of wall height insufficient to affect the retained backfill. Despite having a wall top displacement of 2% at the end of the 0.3g excitation level, as observed when compared to deformation images at the 0.1g excitation level, relatively minimal backfill deformation is visually observed with no apparent failure planes formed at this stage. However, clear settlement of the backfill due to the outward movement of the wall panel can be seen. This is slightly more pronounced at the backfill interface region (where the white reference points for high-speed image capture are located). Similar observations can be made at the 0.4g excitation level with more significant settlement observed. Between the two stages of excitation, the backfill area of significant settlement also increases with significant settlement observed beyond a distance of 1500mm from the wall panel.

At the 0.5g excitation level, significant rotation of the wall panel and thus the reinforced soil block was experienced resulting in a maximum horizontal displacement of 20% of the wall height at the top of the wall panel (Figure 4-13(e)). This was accompanied with a significant sliding displacement...
(maximum incremental increase in all excitation levels) of the wall toe of 34mm or 3.5% of the wall height. Rotation of the reinforced soil block resulted in the development of a failure wedge which had an inclined failure plane that extended from the backfill surface towards the end of the lowest reinforcement layer, R1. Although not clearly visible, another failure surface was formed between the wall face toe and the R1 reinforcement layer. This was also observed in Jackson (2010) and Watanabe et al. (2003) and is further discussed in Chapter 5.

The backfill deformation and wall facing displacement described in this section was typical of all tests in this series of wall models including the surcharged wall models. The effect of different wall parameters between the wall models resulted in different degrees of deformation progression and is presented in the following Sections 4.3.2 to 4.3.8.

### 4.3.2 Cumulative wall displacement

The deformation of a GRS wall would affect its post-earthquake serviceability and therefore, GRS walls should be designed for low permanent deformations under strong shaking. Lateral displacement of the GRS wall provides a useful indication of retaining wall performance. The cumulative wall displacement of the FHR wall panel can be shown as displacement-acceleration curves, where residual displacement after each excitation step is plotted against the input base acceleration. Through these curves, the progression of the FHR wall facing to failure can be determined.

The RMS and PGA acceleration intensity measure can be both used to quantify the input base acceleration (Figure 4-14 & Figure 4-15). In these plots, wall displacement is defined at the displacement of the FHR wall top, at 950mm elevation from the base of the wall. Total wall top displacement ($x_{total}$) can be broken down into two separate components, a rotational component ($x_{rot}$) and sliding component ($x_{slid}$). The latter contribution to wall top displacement is due to horizontal movement of the wall toe whilst the former is due to the rotation of the wall top about the wall toe, calculated as, $x_{rot} = x_{total} - x_{slid}$. Another measure of rotational displacement is the angle of rotational defined as $\theta_{rot} = \tan^{-1}(x_{rot}/H)$ where $H$ is the height of the wall panel.

![Figure 4-14: Displacement-Acceleration Curves for KL5 using RMS acceleration.](image-url)
Similar to Jackson (2010) and El-Emam and Bathurst (2004), a near bi-linear displacement-acceleration curve is observed where beyond a certain threshold input acceleration level, the rate of displacement of the FHR wall increases rapidly indicating failure of the GRS wall.

Prior to the threshold or critical acceleration level, the sliding component of displacement of the wall is relatively small reaching approximately 5mm after the 0.4g excitation level. At this stage, the rotation has contributed to majority of the wall top displacement, 39mm out of 44mm wall top displacement; equivalent to 88% of the total wall top displacement. Beyond the critical acceleration level, the sliding of the wall increases significantly contributing 34mm of displacement or about 18% of the total displacement. Near identical observations were made in Jackson (2010). From these observations, it is apparent that the rotational failure mode of the wall is dominant in all stages of excitation. This is further discussed in the following sections.

Note that in a majority of the following figures in this chapter, the peak base input acceleration is used as the acceleration measure. This is in accordance with majority of published research that have used peak input base accelerations as their acceleration measure (El-Emam and Bathurst, 2007; Tatsuoka, 2008; Watanabe et al., 2003).

4.3.3 Modes of Failure
To examine in greater detail the rotational and sliding components of wall displacement, each component of deformation is non-dimensionalised by dividing each displacement component by the total wall top displacement in each excitation level (Figure 4-16). This enables quantification of the contribution of each component relative to the residual wall top displacement in each excitation step.

As observed in Figure 4-16, the rotational component of wall displacement is pre-dominant in all stages of excitation, contributing to about 80% to 95% of the total wall top displacement during excitation. The sliding contribution to wall displacement decreases initially from 13% to 4% of wall top displacement in the 0.1g to 0.3g excitation levels, but subsequently increases in the 0.4g and
0.5g excitation levels, to 18%. A similar trend is observed in all wall models with the increase in sliding contribution of displacement always observed in the last two excitation levels and an initial decrease in sliding contribution in the first two excitation levels.

In the first excitation step, a high sliding displacement component is observed due to some horizontal displacement needed for engagement of the geogrid reinforcement layers and mobilisation of its resistance. But in the subsequent excitation levels, the pullout resistance of the reinforcement layers reduces the amount of sliding displacement the FHR wall panel can achieve, leading to a decrease in the sliding displacement component. In the later stages of excitation, due to the increased excitation, sliding displacement increases in significance as the frictional resistance of the wall toe and the pullout resistance of the reinforcement layers is exceeded.

**4.3.4 Critical acceleration**

Bracegirdle (1980) defined critical acceleration as “the horizontal pseudo-static acceleration acting uniformly over the structure to achieve limiting equilibrium”. At critical acceleration, the dynamic factor of safety against failure of the wall, typically sliding or rotation, will be less than 1 and permanent displacements will occur. Therefore, critical acceleration is an important measure of GRS wall stability; a high critical acceleration indicates high stability of the wall during an earthquake.

Typically, critical acceleration is defined as the point on the displacement-acceleration curve where accelerations beyond this point result in a sharp increase in lateral displacement (El-Emam & Bathurst, 2007; Jackson, 2010). Similar to Jackson (2010), a bi-linear displacement-acceleration curve is observed in current tests with a sharp increase in sliding displacement observed in the failure excitation level. In Figure 4-15, accelerations larger than 0.4g causes the wall sliding displacement to increase significantly, indicating that the 0.4g excitation level as the model critical acceleration value.

El-Emam & Bathurst (2005) also observed for their 1.0m high wall models that prior to the critical acceleration value, the predominant mode of deformation was wall rotation (Figure 4-17). In a similar manner to Figure 4-17(a), to compare the dominance of each mode of deformation for the
current tests, the rotational and sliding displacement components of the wall are normalised with respect to the wall height of 950mm and plotted together (Figure 4-18). The KL5 wall model demonstrated rotation-dominated behaviour in all levels of excitation with no relative transition from rotational to sliding modes of failure. This differs with El-Emam and Bathurst (2005) that observed a transition to sliding dominated wall displacement at accelerations larger than the critical acceleration (Figure 4-17(a)). This difference in wall behaviour is likely because their model walls’ facing panels were founded on roller bearings of low localised friction (near frictionless wall toe).

However, although rotation-dominated wall displacement was observed for KL5, a significant increase in the sliding contribution of displacement is observed in the excitation level prior to failure. Similar observations were also made by Jackson (2010) who proposed the use of critical acceleration based on this onset of significant sliding. This proposed criterion of critical acceleration is well-suited to the present dataset as a significant increase in the sliding component of displacement typically occurs in the failure excitation level (shown in Figure 4-15).

Focussing on the base of Figure 4-18, there appears to be a critical point where normalised wall sliding is about 0.002 and normalised wall rotation is about 0.015-0.03 (Figure 4-19). Beyond this point, the sliding component of displacement sharply increases as a result of failure of the wall model. This could be linked to the critical acceleration of the wall models, beyond which sliding contribution to displacement increases significantly (Jackson, 2010).

![Figure 4-17: Wall displacement response for sliding toe models: (a) top relative displacement ratio against bottom displacement ratio; (b) top relative displacement ratio against input base acceleration. Note: $\Delta X_b =$ displacement at bottom of wall; $\Delta X_T =$ displacement at top of wall; $H =$ height of wall. (El-Emam and Bathurst, 2005)](image)
Figure 4-18: Normalised sliding and rotational component of wall deformation for KL5.

Figure 4-19: Normalised sliding and rotational component of wall deformation for KL5 (Zoomed In).
4.3.5 Acceleration amplification

Conventional pseudo-static design calculations assume that the backfill experiences a uniform acceleration throughout. Steedman and Zeng (1990) showed that acceleration amplification has an influence similar to the effect of increasing the acceleration coefficient in a uniform acceleration field. This contributes to an increase in the destabilising inertial force acting on the wall.

4.3.5.1 Peak and RMS acceleration

Amplification factor is dependent on the choice of acceleration measure used to calculate the amplification factors. El-Emam and Bathurst (2004) and Nova-Roessig and Sitar (2006) both reported amplification factors based on peak outward accelerations. However, as mentioned previously in Section 4.2.1.1.2, a better acceleration measure to characterise ground motion is the RMS acceleration which includes the effect of amplitude and frequency content of the acceleration record.

The choice of acceleration measure used to calculate the amplification ratio will result in differences in the amplification ratio determined. Amplification factors are determined by normalizing the acceleration measure recorded at some height, h with the base input acceleration measure as shown in (Jackson, 2010):

\[
AF = \frac{PGA_h}{PGA_{base}} \quad or \quad \frac{RMS_h}{RMS_{base}}
\]  

Using the KL5 wall model of 50% relative density as the basis of comparison, the amplification factors determined from PGA and RMS acceleration respectively can be compared.

Figure 4-20: Amplification factors based on PGA for each excitation level for KL5: (a) Reinforced backfill region, (b) Backfill interface region and (c) Unreinforced backfill region.
Figure 4-21: Amplification factors based on RMS acceleration for each excitation level for KL5: (a) Reinforced backfill region, (b) Backfill interface region and (c) Unreinforced backfill region.

By comparing Figure 4-21 and Figure 4-21, we observe that the amplification factors determined through RMS acceleration are smaller than those determined from peak accelerations. Similar observations were also made in Jackson (2010). Furthermore, a greater decrease is observed for the RMS amplification factors at failure. As the RMS acceleration better characterizes the ground motion, this shows that the peak acceleration amplification can overestimate the acceleration response of the wall (Jackson, 2010).

A gradual increase of the acceleration amplification factor with increasing base excitation is observed up to failure which was also reported in El-Emam and Bathurst (2004) and Matsuo et al. (1998). This is consistent in all three locations of the retaining wall backfill; (1) the reinforced backfill, (2) the backfill interface and (3) the unreinforced backfill. However, at the failure excitation level, a decrease in the acceleration amplification factor is observed within the reinforced backfill and backfill interface region. This decrease in amplification factor was also observed in El-Emam and Bathurst (2007) in the backfill interface region. Inversely, the acceleration amplification factor in the unreinforced backfill region is greatest in the failure excitation level. Furthermore, consistent with observations made by Jackson (2010) and Nova-Roessig and Sitar (2006), acceleration amplification in the reinforced region is observed to be non-linear with increasing elevation.

With increasing base excitation and subsequent higher strains within the backfill, the natural frequency of the deposit reduces and approaches the excitation frequency, leading to increased amplification. However, at failure, significant deformation of the reinforced backfill and backfill interface occurs which would have resulted in a much reduced stiffness and high non-linear soil behaviour, preventing the development of large amplifications (Kramer, 1996). Conversely, further increases in acceleration amplification in the unreinforced backfill region are still observed at failure as comparatively there is minimal backfill deformation around the unreinforced backfill accelerometers.
4.3.5.2 Comparisons with other researchers

For the KL4 model of 85% relative density, using RMS acceleration as the acceleration measure, the base input acceleration at the top layer of accelerometers (elevation of 825mm) is amplified by 1.06 in the 0.1g excitation level and increases to a maximum amplification of 1.4 – 1.43 in the 0.5g and 0.6 excitation level (Figure 4-22).

This is similar to values determined from RMS acceleration by Jackson (2010) for a similar model of 89% relative density, where an acceleration amplification of 1.05 and 1.42 was achieved in the 0.1g and 0.5g excitation levels. This indicates good consistency between the two stages of research conducted as both models underwent identical types of excitation.

In El-Emam and Bathurst (2004) (Figure 4-23), models of 86% relative density were excited at 5 Hz with incremental increases of base acceleration of 0.05g at 5-second intervals; similar to model excitation performed in present tests. Acceleration amplification factors were plotted against input base acceleration and it was found that prior to an input base acceleration of 0.4g, the amplification factors were relatively small (between 1.0 to 1.3) but increased significantly thereafter. Similar observations can be made from the KL4 amplification factors determined where acceleration amplification factors increased significantly in the 0.5g excitation level from 1.2 to 1.4 in the backfill interface and 1.3 to 1.5 in the reinforced backfill region (Figure 4-24).

According to elastic theory and assuming a single degree of freedom system with constant damping ratio, frequency and mass, the amplification factor increases as the stiffness of the structure decreases for a frequency ratio, $\omega / \omega_n \leq 1$ (where $\omega_n$ is the structure fundamental frequency) (El-Emam and Bathurst, 2004). At higher excitation levels, the large model deformation reduces the shear modulus of the soil in regions within the backfill thus reducing the overall stiffness of the structure, resulting in high amplification factors.

![Figure 4-22: Amplification factors based on RMS acceleration for the reinforced backfill of KL4 (Dr=85%).](image)
Figure 4-23: Peak outward acceleration amplification factors at selected locations versus input base acceleration amplitude (Dr = 86%) (El-Emam and Bathurst, 2004).

Figure 4-24: Peak acceleration amplification factors for KL4 (Dr = 85%) for (a) the backfill interface (675mm from wall facing) and (b) the reinforced backfill (250mm from wall facing).
Nova-Roessig and Sitar (2006) also conducted shake-table tests on 55% density GRS walls models with wrap-around facing. Amplifications of up to 2.3 for peak input accelerations of 0.15g were reported. The authors also reported that attenuation of peak accelerations by a maximum factor of 0.76 occurred from a base acceleration of 0.46g onwards. Comparatively, a maximum amplification factor of 1.6 was observed for the 50% density models with no attenuation of accelerations occurring (Figure 4-25). A possible explanation of this is the difference in frequency contents of input motions used. Nova-Roessig and Sitar (2006) first used low-amplitude sinusoids to observe elastic behaviour before shaking with a series of 8 to 12 scaled earthquake motions in comparison to a stepped sinusoidal function with a predominant frequency of 5Hz used in present tests.

4.3.6 Earth pressure
Earth pressure cells were positioned to measure the horizontal earth pressures at the rear face of the reinforced soil and the horizontal earth pressures acting on the FHR wall panel. The layout of the earth pressure instrumentation is similar with the instrumentation layout used in Matsuo et al. (1998) with only horizontal earth pressures recorded. A total of four earth pressure cells were located within the backfill at depths of 150mm and 750mm and at the wall panel and backfill interface.

During the testing of the KL6 model, failure of the earth pressure cell located at the end of the bottom reinforcement (Press2) occurred in the failure excitation level with an abrupt cut in pressure reading observed. This was due to an over-extension of the pressure cell cord which resulted in the detachment of the wiring within the pressure cell. This over-extension was likely caused by the entanglement of the pressure cell cable with the reinforcement load cells cables which were laid on top of each other during model construction. Unfortunately, no extra pressure cells were available for the subsequent tests. Therefore, from wall models KL7 to KL12, horizontal earth pressure readings at the end of the bottom reinforcement layer were not recorded.

4.3.6.1 General Trends of Earth Pressure Development
Seismic earth pressure is one of the important external forces acting on retaining walls during earthquakes and is largely affected by the dynamic interaction between the walls and the backfill (Watanabe et al., 2011). In this section, both the static earth pressures and the dynamic earth pressures generated due to model excitation will be presented.
4.3.6.1.1 Theoretical earth pressures expected

During model excitation, the maximum earth pressure will develop when the FHR wall panel moves towards the backfill in relative terms, hence leading to passive soil conditions. However, under static conditions, at the end of each excitation level, the backfill with experience active soil conditions as the wall has rotated and underwent some measure of displacement away from the backfill. Therefore, both the active and passive earth pressure coefficients are used to calculate the theoretical horizontal earth pressures acting on the wall panel.

Using the friction angles determined by Roper (2006) for Albany sand, the earth pressures coefficients can be determined using the formula below:

\[ K_p = \frac{1 + \sin \phi'}{1 - \sin \phi'} = \frac{1}{K_a} \quad (4-4) \]

Where:
- \( K_p \) is the passive earth pressure coefficient;
- \( K_a \) is the active earth pressure coefficient;
- \( \phi' \) is the critical friction angle of the sand.

From the known soil properties presented in Section 3.2.3, we can determine the unit weight of the backfill and the subsequent vertical stresses throughout the depth of the backfill. Using the earth pressure coefficients, the subsequent horizontal stresses can be determined:

\[ \sigma_h' = K_i \times \sigma_v' \quad (4-5) \]

Table 4-3 shows the vertical and horizontal stresses determined for wall models with a backfill relative density of 50%. From Equation 4-4, the active and passive earth pressure coefficients were determined to be 0.32 and 3.12 respectively. The theoretical horizontal earth pressures within the wall model assuming both soil conditions is shown in Figure 4-26.

Table 4-3: Theoretical vertical and horizontal stresses within the GRS wall backfill

<table>
<thead>
<tr>
<th>Depth (mm)</th>
<th>D&lt;sub&gt;r&lt;/sub&gt;=50% Deposit with no surcharge&lt;sup&gt;3&lt;/sup&gt;</th>
<th>Horizontal Stress assuming active conditions (kPa)</th>
<th>Horizontal Stress assuming passive conditions (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Vertical Stress (kPa)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>150</td>
<td>2.34</td>
<td>0.75</td>
<td>7.29</td>
</tr>
<tr>
<td>300</td>
<td>4.67</td>
<td>1.49</td>
<td>14.57</td>
</tr>
<tr>
<td>450</td>
<td>7.01</td>
<td>2.24</td>
<td>21.86</td>
</tr>
<tr>
<td>600</td>
<td>9.34</td>
<td>2.99</td>
<td>29.14</td>
</tr>
<tr>
<td>750</td>
<td>11.68</td>
<td>3.74</td>
<td>36.43</td>
</tr>
<tr>
<td>900</td>
<td>14.01</td>
<td>4.48</td>
<td>43.71</td>
</tr>
</tbody>
</table>

<sup>3</sup>Void ratio, \( e = 0.67 \); Specific Gravity, \( \rho_s = 2.65 \text{ t/m}^3 \); Unit weight, \( \gamma = 15.6 \text{ kg/m}^3 \)
4.3.6.1.2 Static Earth Pressures recorded

The static earth pressure is the earth pressure recorded under static conditions after each excitation level. Due to the outward movement of the wall and deformation of the backfill, the earth pressures recorded within the backfill differed in each excitation level. This is due to tilting of the pressure cells and the reduction in backfill density around the pressure cells as the backfill deforms. Typically, outward movement of the wall away from the backfill will reduce the earth pressures within the backfill as the backfill conditions shifts towards the active state. Furthermore, greater backfill deformation will lead to a reduction in backfill density thus leading to a decrease in horizontal earth pressures. Conversely, the tilting of the pressure cells due to wall displacement will lead to a slight increase in earth pressure recorded, as a greater proportion of vertical earth pressure contributes to the horizontal earth pressures. The residual earth pressures at the end of each excitation level for the KL5 wall model are presented in Figure 4-27.

![Figure 4-26: Theoretical earth pressures at the FHR wall centreline assuming active soil conditions (static) and passive conditions (during excitation).](image)

![Figure 4-27: Residual earth pressure recorded after each excitation level for KL5.](image)
Firstly, it is observed that the measured horizontal earth pressures before commencement of model excitation are in good agreement with the theoretical earth pressures determined assuming active soil conditions. Theoretical earth pressures of 3.7 and 0.75kPa were determined for the Press1&2 and Press3&4 earth pressure cells differing from the measured earth pressures by about 0.25kPa.

Press1, located at the wall toe, exhibits a steady development of residual earth pressures with increasing excitation levels reaching the maximum residual earth pressure of 10kPa (0.86\(\sigma'_{v}\)) at failure. This is likely due to the gradual movement and tilting of the wall (active soil conditions) with increasing excitation levels as observed in wall displacement plots.

There is minimal observed change in the earth pressure recorded at the end of the bottom reinforcement layer (Press2) with a decrease in earth pressure to 3.2kPa (0.3\(\sigma'_{v}\)) observed at failure. This indicates that prior to failure, there was minimal backfill deformation or tilting due to the wall displacement at bottom of the backfill interface. This is in agreement with displacement plots that indicate that sliding displacement only becomes significant at failure. Upon the onset of sliding, pullout of the bottom reinforcement layer occurred which would have led to a disturbance of the soil surrounding Press2.

Press3 and Press4, located along the top reinforcement layer exhibits a gradual increase in residual earth pressure. A drop in residual earth pressure for Press3 at the wall face and the opposite for Press4 at the backfill interface was observed at failure. This is a result of the greater backfill deformation experienced by the top pressure cells compared to the bottom pressure cells, Press1 and Press2. As a result, a muted increase in static earth pressures is observed for Press3 and Press4 compared to the significant increase in earth pressure recorded by Press1 (note that Press2 is largely unaffected by the wall behaviour until failure).

4.3.6.1.3 Maximum Seismic Earth Pressure Recorded within excitation level.
An observed steady rise in seismic earth pressures with increasing excitation levels is recorded in all pressure cells with a large spike in maximum seismic earth pressure typically recorded in the failure excitation level. The maximum dynamic earth pressure development for KL4 and KL5 wall models shown in Figure 4-28 & Figure 4-29 is representative of observed characteristics for all wall models.

Comparing the theoretical horizontal earth pressures determined for passive soil conditions with the dynamic earth pressures recorded within each excitation level, we observe that the passive earth pressures calculated are significantly greater. During excitation, gradual outward displacement of the wall is experienced resulting in active soil conditions (lowest earth pressure). But when the wall panel moves towards the backfill during excitation of the model, passive soil conditions are experienced. However, due to the initial outwards displacement of the wall model, the backfill no longer retains the same properties as it did before excitation (reduced backfill density). As a result, the maximum seismic earth pressures recorded within the backfill is significantly different to the theoretical earth pressures determined for both the passive and active conditions. Maximum seismic earth pressures recorded within each excitation level would be between the theoretical active and passive earth pressures as observed in Figure 4-28 & Figure 4-29.

The Press1 cell (at the wall toe) at a backfill depth of 750mm appears to develop the highest maximum earth pressure with the earth pressure at failure reaching 17kPa (or 1.5\(\sigma'_{v}\)) for the KL5 model. Comparatively, Press3 and Press4 located at the wall and at the end of the top reinforcement
layer (Depth of 150mm) exhibited a maximum earth pressure of 5kPa and 4kPa respectively. Press2, located at the end of the bottom reinforcement layer (Depth of 750mm) exhibited a maximum seismic earth pressure of 10kPa.

Similar earth pressure development is observed for the KL4 model with the exception of greater seismic earth pressures recorded for Press4 compared to Press3. This is in agreement with the general behaviour of the wall models tested where near the surface of the backfill, the earth pressures recorded in the backfill interface is greater than earth pressures recorded at the wall face. This is likely due to the greater relative deformation experienced at the wall face resulting in a greater decrease in backfill density and thus reduction of earth pressures that are developed.

Matsuo et al. (1998) performed shake-table tests on GRS wall models (Dr=60%) and recorded the horizontal earth pressures in at the wall face and at the backfill interface in their research (Figure 4-30). The researchers found that for the continuous facing wall (FHR wall panel), the increase in earth pressure due to shaking is relatively constant with depth with no stress concentration at the
bottom of the wall. A stress concentration at the bottom of the wall was observed for segmental facing walls.

Comparatively, although the present test results do show an increase in earth pressure due to shaking that is constant with depth, the lack of earth pressure readings between the 150mm and 750mm depth limits the conclusions that can be made from present tests (Figure 4-31). More earth pressure cells placed at differing depths are needed to classify the earth pressure distribution in a similar manner to Matsuo et al. (1998).

Figure 4-30: Maximum earth pressure during each excitation step: (a) Discrete facing panels (b) Continuous facing (Matsuo, 1998).

Figure 4-31: Maximum earth pressure distribution at the wall face and backfill interface for KL5.
4.3.7 Reinforcement loads
Wall models KL4 to KL12 were instrumented with load cells to determine the reinforcement loads generated within the model due to excitation. Three out of five reinforcement layers were instrumented with load cells (total of 9 load cells) due to the possibility that the load cell attachment might influence the reinforcement behaviour and thus affect the general behaviour of the GRS wall. These reinforcement layers are R1, R3 and R5 located at elevations of 150mm, 450mm and 750mm respectively. Further information regarding positions of load cells can be found in Figure 3-9 and Table 3-6 in Chapter 3. As stated previously, positive reinforcement loads refer to tension experienced by the load cells within the reinforcement layer and negative reinforcement loads refer to compression of the load cells.

4.3.7.1 Reinforcement load distribution within reinforcement layer
For the 0.1g and 0.2g excitation steps, the centreline load cells developed greater incremental increases of tensile load in comparison to the two adjacent load cells; the North load cells and South load cells (Figure 4-32). Similarly, for higher excitation levels, where a reduction in reinforcement loads are observed for the R3 reinforcement layer, the centreline load cells exhibited a greater incremental decrease of reinforcement load. These observations show that the centreline load cell typically exhibits a pronounced/greater magnitude of reinforcement load decrease or increase in each excitation level. In other words, the centreline load cells in the reinforcement layers appear to be highly dominant in terms of reinforcement load distribution.

![Figure 4-32: Incremental reinforcement load development in the: (a) 0.1g excitation level and (b) 0.4g excitation level.](image)
The dominance of the centreline load cell in reinforcement load distribution can be explained by a simple force diagram (Figure 4-33). By assuming the geogrid steel plate as a two-span beam with free ends under uniform load and with the load cells representing pinned-supports, we can determine the load distribution between the load cells through moment-area theorem. Theoretical analyses show that the centreline load cell will take up approximately 50% of the total reinforcement load (UDL x length of steel strip) with the adjacent load cells attracting 25% of total reinforcement load respectively.

![Figure 4-33: Illustration of reinforcement load distribution within reinforcement layer.](image)

Table 4-4 shows the distribution of reinforcement load development in percentages for each instrumented reinforcement layer for wall model KL5 and as an average of all wall models tested. The incremental load development under static conditions is not used due to installation effects on reinforcement loads present.

It is observed that the experimental results are generally in agreement with theoretical analysis with reinforcement load evenly distributed on either side of the centreline and the Centreline load cell generally displaying a greater reinforcement load proportion. Discrepancies could be due to (i) the non-uniform FHR wall panel displacement during excitation about the centreline and (ii) slight differences in plan area of geogrid reinforcement on either side of the centreline (resulting in a non-uniformly distributed load).

**Table 4-4: Distribution of reinforcement load development (%) within each instrumented reinforcement layer for KL5 and averaged for all wall models**

<table>
<thead>
<tr>
<th></th>
<th>R1 Reinforcement Layer (Elevation: 150mm)</th>
<th>R3 Reinforcement Layer (Elevation: 450mm)</th>
<th>R5 Reinforcement Layer (Elevation: 750mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>North</td>
<td>Centre-line</td>
<td>South</td>
</tr>
<tr>
<td>KL5 – 0.1g Excitation</td>
<td>20</td>
<td>55</td>
<td>25</td>
</tr>
<tr>
<td>KL5 – 0.4g Excitation</td>
<td>29</td>
<td>28</td>
<td>44</td>
</tr>
<tr>
<td>Averaged of all models – 0.1g Excitation</td>
<td>22</td>
<td>58</td>
<td>19</td>
</tr>
<tr>
<td>Averaged of all wall models – 0.4g Excitation</td>
<td>29</td>
<td>36</td>
<td>36</td>
</tr>
</tbody>
</table>
4.3.7.2 General Progression of reinforcement load development

4.3.7.2.1 Reinforcement load development within excitation level

For all excitation steps, the reinforcement load development within the excitation level is similar to the dynamic motion of excitation, sinusoidally increasing or decreasing with the same frequency. However, different in reinforcement load development within the failure excitation step were observed in comparison to previous excitation steps (Figure 4-34).

![Diagram showing reinforcement load development](image)

Figure 4-34: Reinforcement Load Development of (a) R1, (b) R3 and (c) R5 reinforcement layer in 0.6g excitation level for KL5.
As observed in Figure 4-34(a), the bottom reinforcement layer, R1 experiences an initial increase in reinforcement load before sharply decreasing, resulting in a reduction of reinforcement load accumulated at the end of the excitation step. This sudden drop in accumulated reinforcement load is likely to be due the pullout failure of the bottom geogrid layer, leading to an increase in sliding displacement and the eventual failure of the retaining wall. Reinforcement load development for the R1 layer is similar to that observed in 300mm by 300mm direct shear tests of conducted by Alobaidi et al. (1997), where the normalised shear stress reaches a peak before decreasing gradually to a critical shear stress level with increased horizontal displacement.

Additionally, a distortion of accumulated reinforcement load is observed at the end of the excitation step observed in all reinforcement layers. In most load cell readings (except Load7 and Load8), a decrease in the amplitude of the subsequent reinforcement load cycles is observed. This is due the wall facing reaching the limit of displacement, eventually coming to rest on the external support. As the FHR wall panel is unable to displace further, the reinforcement load development is limited, explaining the decrease in amplitude of the load cycles. Furthermore, as the FHR wall panel is at rest on the external support, some of the reinforcement load is taken up by the external support. The residual reinforcement load at the end of the failure excitation step is less than the load experienced by the reinforcement layers at failure, hence the apparent decrease in accumulated reinforcement load in the last few cycles. The reinforcement load experienced at failure is of interest as it enables the determination of the total load applied to the reinforcement layer before failure. To determine this failure reinforcement load, the accumulated reinforcement load recorded in the last load cycle before the sudden decrease is averaged as illustrated in Figure 4-35.

Figure 4-35: Illustration of the determination of the failure reinforcement load.
4.3.7.2.2 Reinforcement load development within GRS wall model (along wall elevation)

After removal of the FHR wall panel struts before shaking, the reinforcement loads developed are influenced by both the internal interaction between the steel plate and the FHR wall panel and the outward rotation of the FHR panel under static conditions. Upon removal of the wall panel struts, relatively similar increases in reinforcement load were observed between the wall models ranging from 0.3kN/m to 0.35N/m (Figure 4-36).

For the 0.1g and 0.2g excitation levels, due to the outward movement of the retaining wall, each reinforcement layer experiences an increase in reinforcement load, indicating the development of tensile forces. A near linear reinforcement load profile develops at these two excitation levels with greater reinforcement loads experienced by reinforcement layers at greater depth. Incremental load development at the 0.2g excitation level was determined to be 0.27kN/m in R1, 0.08kN/m for R3 and 0.005kN/m for R5. Greater incremental increase in reinforcement load for the lower R1 reinforcement layer is due to the relatively larger normal stress experienced by the reinforcement layer.

From the 0.3g excitation level onwards, a clear distinction in reinforcement load development between the three instrumented reinforcement layers is observed. At the 0.3g excitation level, an incremental decrease in reinforcement load is experienced by the middle reinforcement layer, R3, and incremental increases in reinforcement loads for the R1 and R5 layers. The reduction in reinforcement load at R3 becomes more pronounced at higher excitation levels until failure. It is important to note that for the wall models that failed at the 0.6g excitation level (LH09 wall models), this decrease in reinforcement load is slight or non-existent for the 0.3g excitation level but becomes evident in the 0.4g excitation level. This is because the FHR wall progression to failure is not as far along as the other wall models at this stage of excitation; therefore, the models show a delayed reinforcement load progression comparatively.
For the excitation level at which wall failure occurred (at 0.5g for StagLH and LH075 wall models and 0.6g for LH09 models), a reduction of reinforcement load develops in the lowest reinforcement layer, R1. As explained previously in Section 4.3.7.2.1 and observed in Figure 4-34(a), this reduction in reinforcement load for R1 is due to the pullout failure of the reinforcement layer, leading to a decrease in reinforcement loads recorded. Incremental increases in reinforcement load are still observed for the R3 and R5 reinforcement layers. As both reinforcement layers did not fail from pullout failure, the outward displacement of the FHR panel in the failure excitation step results in positive reinforcement load development.

4.3.7.2.3 Reinforcement load development between instrumented layers

For all wall models, accumulation of reinforcement load for each reinforcement layer exhibits a similar trend as shown in Figure 4-37.

The lowest reinforcement layer, R1 exhibits a gradual accumulation of reinforcement load in all excitation levels up till wall failure where a decrease in accumulated reinforcement load is observed. The R1 reinforcement layer develops the largest reinforcement load in comparison with the R3 and R5 reinforcement layer. The sudden decrease in reinforcement load recorded in the failure excitation level is due to the pullout failure of the R1 reinforcement layer (reduced pullout resistance due to failure leading to reduced reinforcement load). This is also related to the significant increase of the sliding component of wall displacement at failure.

For the R3 reinforcement layer, for the 0.1g and 0.2g excitation levels, incremental increases in reinforcement load are experienced but with decreasing magnitude. For the 0.3g excitation level onwards, a reduction of reinforcement load is observed (compressive forces are incrementally applied to this reinforcement layer during 0.3g and 0.4g excitation level). At failure, positive reinforcement load development is again observed. This initial peaking of accumulated reinforcement load is further investigated in Section 4.3.7.2.3.

![Figure 4-37: Accumulated Reinforcement Load (kN per unit width) for KL5 between instrumented reinforcement layers.](image-url)
For the R5 reinforcement layer, a gradual accumulation of reinforcement load is observed with greater increments of tensile load developed in the later stages of excitation (0.4g onwards). Consistent in all wall models, the 0.2g excitation level did not result in any significant changes to reinforcement load (incremental reinforcement load approximately zero). The larger increments of reinforcement load observed in the 0.4g excitation level onwards is likely due to the larger wall top displacement increments experienced in these excitation levels.

Comparing all three reinforcement layers’ load development, a transfer of reinforcement load from the R1 reinforcement layer to the R3 and R5 reinforcement layer at failure becomes apparent. This is likely related to the pullout failure of the R1 reinforcement layer leading to greater loads taken up by the other reinforcement layers.

**4.3.7.2.4 Relationship with Wall Displacement**

The influence of wall displacement on the reinforcement loads developed can be analysed through wall displacement – reinforcement load plots. After each excitation level, the residual wall displacement at each instrumented reinforcement layer is determined (Elevations of 150mm, 450mm and 750mm) and plotted with the reinforcement load developed in the reinforcement layer (Figure 4-38). Reinforcement load – wall displacement plots for each reinforcement layer are similar between the wall models tested indicating good agreement for reinforcement load development.

For the bottom reinforcement layer, R1, reinforcement load development increases steeply with wall displacement before reaching a peak reinforcement load at about 10-20mm horizontal displacement. This peak reinforcement load is typically experienced during the last two excitation levels prior to the failure excitation level (eg. 0.4g and 0.5g excitation level for LH09 wall models). Beyond this peak value, a significant increase in horizontal displacement led to a decrease in accumulated reinforcement load. This once again indicates the pullout failure of the reinforcement that led to an increase to sliding displacement observed with a decrease in reinforcement load.

![Figure 4-38: Reinforcement load – horizontal reinforcement displacement plots for KL5.](image)
Reinforcement loads developed in the middle R3 reinforcement layer peaked at a horizontal wall displacement of 10mm before sharply decreasing and reaching its lowest accumulated reinforcement load (excluding static load) of 70% of peak reinforcement load. However, at the failure excitation level, the reinforcement load recorded increases to a similar magnitude as the peak reinforcement load. The initial peaking of the R3 reinforcement load is observed despite increased reinforcement displacement and is observed in all wall models. This indicates possible pullout failure of the R3 reinforcement layer or slips in frictional resistance at the soil-geogrid interface as an incremental decrease in reinforcement load is experienced. However, more studies should be done to ensure that this is not a result of the experimental setup used in this series of GRS wall models.

Initially for the R5 reinforcement layer, an apparent plateau of accumulated reinforcement loads is observed within 10mm of wall displacement. However, beyond 10mm of horizontal displacement, subsequent increases in accumulated reinforcement load are observed up to failure. Note that despite the R5 reinforcement layer experiencing gradual development of reinforcement load throughout excitation and the greatest magnitude of horizontal displacement, reinforcement loads developed were still less than that achieved in the bottom R1 reinforcement layer. This is likely a function of greater vertical stresses experienced in the R1 reinforcement layer, hence a greater peak reinforcement load.

4.3.8 Phase Differences

4.3.8.1 Reinforcement Loads Development
Within each reinforcement layer, the three load cells in a single reinforcement layer experience no phase differences in reinforcement load development as indicated in Figure 4-39.

Between reinforcement layers, reinforcement load development of all reinforcement layers is initially in-phase but load development for the R5 reinforcement layer gradually progresses to be out-of-phase at the higher excitation levels (Figure 4-40 & Figure 4-41).

![Figure 4-39: Reinforcement Load development within the 0.5g excitation level for KL7.](image)
Figure 4-40: Reinforcement Load Development for each reinforcement layer within the 0.3g excitation level for KL7.

Figure 4-41: Reinforcement Load Development for each reinforcement layer within the 0.5g excitation level for KL7.
4.3.8.2 Reinforcement Load vs wall displacement

As reinforcement load development and wall displacement are closely related to one another, the cyclic development of the two parameters is of interest to determine if any phase differences exist. Cyclic horizontal displacements of individual reinforcement layers are determined at each reinforcement elevation and compared with reinforcement load development by plotting both parameters against time in Figure 4-42. However, as the amplitude of the cyclic reinforcement load development is larger than that of cyclic wall displacement, the reinforcement loads shown below is scaled-down by the maximum reinforcement load recorded within the excitation level. This is done to enable clear comparisons of peaks between the cyclic reinforcement load and cyclic wall displacements developed within the same plot. Similarly, displacements recorded were also scaled-down by the maximum displacement recorded within the excitation level and multiplied to increase the amplitude of each displacement cycle; ensuring clear presentation of each cycle.

Figure 4-42 shows that there is a slight phase difference between the reinforcement load development and the FHR wall displacement with reinforcement load development reaching a peak before cyclic wall displacement. The earlier peaking of reinforcement loads is due to the reinforcement layers initially resisting the outward displacement of the wall through its pullout resistance. After each loading cycle peak, some magnitude of reinforcement layer displacement is experienced shown by the corresponding reinforcement displacement peak. This indicates that some slippage of the reinforcement is experienced within each loading cycle corresponding to a residual displacement of the reinforcement recorded at the end of the excitation level.

![Figure 4-42: Cyclic development of wall displacement and reinforcement load (scaled-down) during the 0.5g excitation level for KLS.](image)
4.4 Influence of GRS wall parameters on seismic performance

In this section, the influence of reinforcement length and layout, soil density, and surcharging on the seismic performance of the GRS walls is investigated. To do so, methods of analysis developed in Section 4.3 are used to compare between the different wall models. Reasons for the discrepancies between the tests are also discussed.

4.4.1 Accumulation of residual displacement

Previous researchers have found that the cumulative wall displacement is influenced by the reinforcement length, soil density, and surcharging (El-Emam and Bathurst, 2007; Jackson, 2010). By comparing the residual displacement-acceleration curves, each parameter’s effect on wall facing displacement can be determined. The displacement-acceleration curve for the non-surcharged and surcharge wall models are shown in Figure 4-43 and Figure 4-44.

4.4.1.1 Influence of reinforcement length and layout

As discussed in Chapter 3, reinforcement length was varied between the models of 50% backfill density with three separate reinforcement layouts tested: (i) Uniform reinforcement layout of L/H = 0.75 (LH075), (ii) Uniform reinforcement layout of L/H = 0.9 (LH09) and (iii) Staggered reinforcement layout with the top two reinforcement layers at L/H = 0.9 and the bottom three at L/H = 0.75 (StagLH).

Both Figure 4-43 and Figure 4-44 clearly show the direct relationship between reinforcement length and wall stability with the KL6 and KL7 wall models (both LH09) having shallower displacement-acceleration curves and a higher critical acceleration triggering failure than the KL5 (LH075) wall model, which had a shorter uniform reinforcement length. This is in agreement with results of Jackson (2010).

Figure 4-43: Cumulative Displacement - Acceleration curves for non-surcharged wall models.
Figure 4-44: Cumulative Displacement - Acceleration curves for surcharged wall models.

For staggered reinforcement layout models (StagLH), the longer reinforcement provided as the top two layers is observed to have increased the wall stability. This is evident in the shallower displacement-accelerations plot of KL8 (non-surcharged) in Figure 4-43 and that of KL12 (surcharged) in Figure 4-44 compared to the uniform LH075 reinforcement models, KL5 and KL11 (non-surcharged and surcharged, respectively). The increase in the top two reinforcement layer lengths by 20\% resulted in a reduction of wall displacement by 23\% (non-surcharged) and 17\% (surcharged) in the excitation level prior to failure. However, although the longer reinforcement length increased the stability of the wall, the staggered reinforcement wall models still failed at the 0.5g excitation level similar to the LH075 models. This is because the staged excitation of the model with increments of 0.1g was too coarse to pick up the small differences between LH075 models and StagLH models.

4.4.1.2 Influence of soil density

To determine the effect of soil density on GRS wall behaviour, we compare the KL4 and KL5 wall model responses (Figure 4-45). The KL4 wall model was constructed in an identical manner to KL5 but with a backfill relative density of 85\% compared to 50\% in KL5.

The KL4 model exhibited much higher stability, reaching failure at the 0.6g excitation level, compared to KL5 which failed at 0.5g. The displacement-acceleration curves show that at all excitation levels, the 50\% density model (KL5) exhibits a greater wall displacement than the 85\% density KL4 model. Note that the KL4 displacement-acceleration curve is shallower compared to the KL6 and KL7 models which were LH09 models with 50\% relative density and also failed in the 0.6g excitation level (Figure 4-45). This indicates that an increase in backfill density by 35\% has a greater degree of GRS wall stabilisation compared to an increase of the model reinforcement length by 20\%.

This agrees well with theoretical understanding of the soil-geogrid interaction (Jewell et al., 1985) that the increase in soil density would lead to greater soil shear resistance as well as soil-geogrid shear resistance. Also, as dense soils dilate more than loose soils, the dilatant behaviour of the dense sand caused by pullout of the reinforcement is confined, leading to an increase in normal stress around the geogrid and thus increasing the soil-geogrid interface strength (Alfaro et al., 1995).
The results of present tests are in good agreement with Lopes and Ladeira (1996) who performed pullout tests of geogrid in samples of different relative density, 50% and 86%. They found that for the looser sample, the geogrids failed by lack of adherence between the geogrid and sand, resulting in the pull-out of the geogrid reinforcement. For the denser sample, the soil-reinforcement interface maintained its integrity and the reinforcement failed through a lack of tensile strength. Failure of the reinforcement through a lack of tensile strength is attributed to their high confinement pressure of 46.7 kPa which is about 4 times the maximum confining pressure (12.5kPa) within the 85% relative density model at the bottom R1 reinforcement layer.

**4.4.1.3 Influence of 3kPa surcharge**

In the initial stages of excitation (0.1g to 0.3g), the surcharged models generally exhibit a lower displacement-acceleration curve compared to the non-surcharged models, implying that the surcharge reduces the GRS wall displacement in the initial excitation levels (Figure 4-46). However, in the 0.4g excitation level, a larger accumulated wall top displacement is recorded by the surcharged models, indicating a greater incremental increase in wall displacement is experienced in comparison to the non-surcharged models (Figure 4-47). At failure, the surcharged LH075 and StagLH models exhibits a reduced wall top displacement compared to their non-surcharged counterparts. However, the opposite was observed for the LH09 models with the surcharged KL10 model exhibiting a greater wall top displacement than KL6 and KL7.

The resulting wall behaviour is a function of several interactions between the surcharge and the GRS wall model. In the lower excitation levels the effect of the surcharge was to reduce the wall displacement due to the increased pullout resistance due to additional confinement. However at higher excitation levels, the surcharge acts to add to the dynamic loading of the GRS wall, destabilising the GRS wall.

One effect of the backfill surcharge would be to increase soil stresses within the backfill leading to an increase in normal stresses experienced by each individual reinforcement layer. Therefore, the surcharge would increase the soil-geogrid interaction; increasing the pullout resistance of the
reinforcement layers thus increasing wall stability. Note that the degree of normal stress increase diminishes with depth from the surface of the backfill as shown previously in Chapter 3.

However, conversely, a surcharge may also increase the active earth pressure acting on the retaining wall, acting as a destabilizing force. Furthermore, at higher excitation levels, the surcharge may act to increase the dynamic loading of the GRS wall by increasing the shear stresses within the backfill due to its potential for out-of-phase motion relative to the backfill due to its own inertia. At the formation of the failure wedge, the surcharge on the backfill within the failure wedge becomes a contributor to the destabilizing forces of the failure block as it applies additional vertical stresses to the failure block. These increased shear stresses would act on the GRS wall further destabilising it under dynamic loading.

Figure 4-46: Displacement-acceleration curves for surcharged and non-surcharged models in the initial stages of excitation (0.1g to 0.3g).

Figure 4-47: Displacement-acceleration curves for surcharged and non-surcharged models.
The greater incremental increase in wall top displacement for the surcharged models in the 0.4g excitation level (Figure 4-47), is likely due to the greater contribution of the surcharge to the destabilising force that its contribution to the resisting forces of the GRS wall, leading to destabilization of the wall. The onset of the greater contribution of the surcharge to the wall destabilising forces likely coincides with the development of the first failure plane formed in the backfill. At the formation of the first failure plane, it is likely that the reinforcement layers within the failure plane will play a reduced role in the stability of the wall as the layer becomes part of the failure block; indicated by the increases in wall displacement observed in the last two excitation levels.

4.4.2 Modes of failure
Research by Jackson (2010) observed that the reinforcement length ratio had minimal effect on the modes of failure. In the following sections, the influence of reinforcement length is further investigated along with the influence of reinforcement layout, backfill density and surcharging on the modes of failure.

4.4.2.1 Influence of reinforcement length and layout
To determine the influence of the reinforcement length on the modes of failure, the contribution of each displacement component to total displacement is compared between the wall models. As the sliding and rotational component of wall displacement is inter-dependent (ie. their sums equal to 1), only sliding contribution of each wall model is shown in the following figures for comparison.

From Figure 4-48 and Figure 4-49, we observe that the sliding contribution to wall top displacement reduces initially from excitation levels 0.1g-0.3g and subsequently increases at excitation levels 0.4g-0.5g with failure occurring at 0.5g for the StagLH and LH075 models. For the LH09 models, increase in sliding contribution to wall top displacement commences in the 0.5g excitation level with failure of the wall occurring at 0.6g. This trend is consistent with all wall models performed with and without surcharge.

For the non-surcharged models, the LH09 wall models exhibit similar sliding contribution ratios compared to the LH075 and StagLH wall models in the later excitation levels. The sliding component ratio for the LH09 models in the 0.4g excitation level is equivalent to the ratio determined in the 0.3g excitation level for the LH075 and StagLH models, with similar observations made for the ratios determined in the 0.5g and 0.6g excitation levels. This observed delay in sliding contribution development is because of the greater resistance of the LH09 wall models resulting in a delayed wall displacement development. However, these observations do not apply to the surcharged LH09 model due to the influence of the backfill surcharge on the modes of failure (discussed further in the Section 4.4.2.3).

Prior to failure, in the initial excitation levels, the KL8 and KL12 StagLH models exhibit slightly higher sliding contribution to wall displacement compared to KL5 and KL11 LH075 models but a lower sliding contribution in the 0.4g excitation level (Figure 4-48 & Figure 4-49). However, at failure, the StagLH wall models experience a greater incremental increase of sliding displacement with sliding accounting for 22% of the total wall displacement for KL8 compared to 18% for KL5 (about 7mm difference of wall toe displacement). Similar observations can be made for the surcharged models albeit to a lesser degree.
To further investigate the effect of the extended two top reinforcement layers in the StagLH wall models, we compare the incremental increases in sliding and rotational displacement between the LH075 and StagLH wall models (Figure 4-50 & Figure 4-51).

Note that prior to failure, the KL8 (StagLH) model exhibits lower sliding and rotational displacement components at each excitation level compared to the KL5 (LH075) model evident in the residual wall top displacement plots (Figure 4-43). Therefore, at failure, we would expect the incremental increase of displacement components for KL8 to be greater as the wall will have to displace more to reach the failure point within the final excitation step (for the wall panel to rest on the external support). From Figure 4-51, the incremental increase of rotational displacement for both the StagLH and LH075 wall models at the failure excitation level is near identical but the incremental sliding displacement increase of the StagLH wall models is greater compared to the LH075 wall models as shown in Figure 4-50.
This explains the previous observations made from Figure 4-48 and Figure 4-49, where the greater sliding component contribution in the failure excitation step is observed. The longer top two layers of reinforcement in the StagLH wall models likely restricted the rotation of the wall under dynamic loading and thus the wall had to slide more to reach the failure displacement (i.e. to rest on the fixed support). This is further confirmed by comparing the angle of rotation of the wall for KL8 and KL5 (Figure 4-52). We observe that KL5 has a higher angle of rotation compared to KL8 at all excitation levels with similar observations for the surcharged wall models KL11 (LH075) and KL12 (StagLH).
4.4.2.2 Influence of soil density

To investigate the influence of soil density on the GRS wall modes of failure, wall top displacement ratios of wall models KL4 and KL5 which had backfill relative densities of 85% and 50% are compared in Figure 4-53.

Figure 4-52: Angle of FHR wall panel rotation for LH075 and StagLH wall models.

Figure 4-53: Sliding and rotational component of displacement for KL4 and KL5 wall models.
Figure 4-53 show that both wall models show an increase in the sliding component of displacement in the excitation level two steps prior to failure (the 0.3g and 0.4g excitation level for KL5 and KL4 respectively) with a similar sliding contribution to wall displacement achieved at failure. The lower density, KL5 model does show a slightly greater increase in sliding in the excitation step prior to failure at about 11% of wall top displacement compared to 8% for the KL4 model.

The difference in displacement components between the two models could be due to a greater overturning force applied to the FHR wall panel as the failure wedge forms (greater soil density results in a larger failure wedge mass and thus greater forces applied by the failure wedge). That said, the difference in displacement components observed is very slight and conclusions are difficult to be made based on comparisons of two models. Further tests should be performed to increase the data pool and thus confirm the influence of soil density on the global deformation pattern of the GRS wall.

### 4.4.2.3 Influence of 3kPa surcharge

Consistently for the StagLH and LH075 wall models, a greater sliding contribution is observed in the surcharged models in the failure excitation level (Figure 4-54). This is further supported by the higher and lower incremental increase of sliding and rotational displacement respectively at the failure excitation step as observed in the incremental displacement component plots of Figure 4-50 and Figure 4-51. This indicates that for the StagLH and LH075 tests, while rotation was still the dominant mode of failure, the effect of surcharge was to decrease the rotational contribution to total wall displacement.

![Figure 4-54: Sliding component contribution plot for StagLH and LH075 models.](image-url)
Comparatively, in Figure 4-55, the KL10 (LH09) model under 3kPa surcharge exhibited a much reduced ratio of sliding displacement, 0.13 compared to the KL6&7 model of 0.22 at failure. This indicates that at failure the KL10 model with surcharge underwent a greater degree of rotation as compared to its non-surcharged counterparts. This is opposite to what is observed for the LH075 and StagLH tests.

Differences between the models can be explained by analysing the mechanisms caused by the presence of backfill surcharge mentioned in the previous Section 4.4.1.3. It is apparent that at failure for the StagLH and LH075 models, the stabilising effect of the surcharge still remains dominant reducing the displacements of the top reinforcement layers, thus reducing the rotational component of wall displacement. However for the longer reinforcement LH09, model, the failure wedge formed during the final excitation level would have been larger than that formed in the shorter reinforcement models. Due to this larger failure wedge, a larger proportion of the surcharge load would be included in the failure wedge, adding to the destabilising overturning force acting on the wall.

Therefore, the LH09 models would have a larger destabilising force acting on the wall compared to the LH075 and StagLH models. At the onset of the failure plane, it is likely that the contribution of the surcharge to the destabilising force was greater than its contribution to the stabilising forces, resulting in greater rotational displacement for KL10-LH09 surcharged model compared to the non-surcharged, KL6 & 7 models.

This is further supported in incremental rotational displacement plots (Figure 4-56) that show that the surcharged LH09 model exhibits a lesser incremental increase in rotational displacement in the 0.1g to 0.3g excitation levels, only displaying a greater incremental increase in the 0.4g onwards. It is likely that this coincides with the formation of the first failure plane in the 0.4g excitation level.
4.4.3 Critical acceleration

As stated previously in Section 4.3.4, the critical acceleration of a GRS wall model is defined as the point beyond which a significant increase in lateral wall displacement occurs. In Figure 4-57, significant increases in lateral wall displacement is observed in the last excitation level indicating that the critical acceleration for each wall model is between the last two excitation levels. Due to the coarseness of model excitation levels, the critical acceleration for each wall model is determined to the nearest 0.1g (taken as the excitation level prior to failure) as shown in Table 4-5. For this reason, the critical accelerations determined for the StagLH and LH075 wall models are identical.

Figure 4-56: Incremental rotational displacement development for the LH09 models.

Figure 4-57: Sliding Component of displacement for all wall models.
Table 4-5: Critical Acceleration levels for each wall model

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Reinforcement Layout</th>
<th>Density</th>
<th>Surcharge</th>
<th>Critical Acceleration Level (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>KL4</td>
<td>0.75</td>
<td>85%</td>
<td>0.5</td>
<td></td>
</tr>
<tr>
<td>KL5</td>
<td>0.75</td>
<td>50%</td>
<td>0.4</td>
<td></td>
</tr>
<tr>
<td>KL6</td>
<td>0.9</td>
<td>50%</td>
<td>0.5</td>
<td></td>
</tr>
<tr>
<td>KL7</td>
<td>0.9</td>
<td>50%</td>
<td>0.5</td>
<td></td>
</tr>
<tr>
<td>KL8</td>
<td>Top two layers @ 0.9 Rest @ 0.75</td>
<td>50%</td>
<td>0.4</td>
<td></td>
</tr>
<tr>
<td>KL9</td>
<td>Top two layers @ 0.9 Rest @ 0.75</td>
<td>50%</td>
<td>3 kPa</td>
<td>0.4</td>
</tr>
<tr>
<td>KL10</td>
<td>0.9</td>
<td>50%</td>
<td>3 kPa</td>
<td>0.5</td>
</tr>
<tr>
<td>KL11</td>
<td>0.75</td>
<td>50%</td>
<td>3 kPa</td>
<td>0.4</td>
</tr>
<tr>
<td>KL12</td>
<td>Top two layers @ 0.9 Rest @ 0.75</td>
<td>50%</td>
<td>3 kPa</td>
<td>0.4</td>
</tr>
</tbody>
</table>

As observed in Table 4-5, there appears to be minimal influence of backfill surcharging on the critical acceleration of the wall models as both surcharged and non-surcharged models have similar critical acceleration values. Conversely, the reinforcement length and backfill density strongly influence the critical acceleration of the wall models with a longer uniform reinforcement length and higher backfill density contributing to a greater critical acceleration level. It is noted that the increase of reinforcement length for the top two layers of reinforcement in the staggered reinforcement models appear to have minimal effect on critical acceleration as well. However, this is likely due to the coarseness of the level of excitation of the wall models as the staggered reinforcement model showed lower wall displacement than the uniform LH075 model, indicating higher wall stability. A more refined progression of model excitation magnitude could have resulted in a higher critical acceleration value being determined for the staggered reinforcement models.

4.4.4 Acceleration amplification

Previous studies have shown that acceleration amplification is dependent on several factors such as soil density, location in the backfill, wall facing and magnitude of base input acceleration (El-Emam and Bathurst, 2005, 2007; Nova-Roessig and Sitar, 2006; Jackson, 2010). However, conflicting results about the influence of reinforcement properties (length and stiffness) were found by Nova-Roessig and Sitar (2006) and El-Emam and Bathurst (2007). In the following sections, the influence of such parameters on the acceleration amplification is discussed.

4.4.4.1 Influence of reinforcement length and layout

Acceleration amplification within the backfill is influenced indirectly by the reinforcement length through the wall facing displacement. This is confirmed in the unreinforced backfill acceleration amplification factors (Figure 4-58) that show relatively similar amplification factors between the wall models at lower elevations with the LH09 models developing comparatively higher amplification factors at the elevation of 825mm. Accelerometers placed in the unreinforced backfill are assumed to represent the “far-field” conditions and are independent of wall displacement. Thus, to compare the influence of reinforcement length on acceleration amplification, amplification factors based on accelerations recorded within the reinforced backfill for surcharged models are compared in Figure 4-59 which is representative of both the surcharged and non-surcharged models as both models show similar acceleration amplification development.
We observed that all wall models developed a peak amplification factor in the excitation level prior to failure. The wall models that failed in the 0.5g excitation level (LH075 and StagLH) develop similar amplification factors within the reinforced backfill with the StagLH models exhibiting slightly lower amplification factors in both the surcharged and non-surcharged models. However, the LH09 models display a delayed development of amplification factor compared to the LH075 and StagLH models (lower amplification factors are observed). This is likely due to their reduced backfill deformation as a result of greater wall stability. Generally, the LH09 models develop lower amplification factors compared to the StagLH and LH075 models in the excitation levels leading up to failure with the difference more pronounced at higher elevations. These observations indicate that the acceleration amplification factors decreases with increasing overall GRS reinforcement length.

Figure 4-58: RMS amplification factors for the Unreinforced backfill (far-field) region for the surcharged models: (a) Blue = Elevation of 825mm; (b) Green = Elevation of 525mm; (c) Red = Elevation of 225mm.

Figure 4-59: RMS amplification factors for the Reinforced backfill region for the surcharged models: (a) Blue = Elevation of 825mm; (b) Green = Elevation of 525mm; (c) Red = Elevation of 225mm.
Results of El-Emam and Bathurst (2007) (Figure 4-60) show good agreement with the current tests with lower amplification factors determined for longer reinforcement models in excitation steps prior to critical acceleration as well. Note that observations from both the present tests and El-Emam and Bathurst (2007) conflicts with studies of centrifuge-tested wrap-around facing wall models by Nova-Roessig and Sitar (2006) who found similar amplification factors for wall models with different reinforcement lengths and thus concluded that amplification is not strongly influenced by reinforcement length or stiffness.

The differences between the different reinforcement length wall models tested is likely due to the amount of model deformation experienced by the wall model at each excitation level. The longer reinforcement models will undergo less model deformation at a given excitation level due to its higher stability. Thus, there is less shear deformation within the backfill resulting in a higher shear modulus and higher overall stiffness of the structure, maintaining its natural frequency. The shorter reinforcement models which experiences larger model deformations and a reduced stiffness of the structure and thus, high amplifications according to elastic theory (El-Emam and Bathurst, 2004). The natural frequency of the deposit (14.8Hz) will also decrease with higher backfill strains and tend towards the lower excitation frequency of 5 Hz. Furthermore, the shorter reinforcement of the wall models could be encouraging some rocking movement about the wall toe during excitation thus contributing to amplification with the reinforced backfill.

4.4.4.2 Influence of soil density

As mentioned in the previous section, model deformation influences the acceleration amplification within the backfill. To isolate the effect of soil density on the acceleration amplification, the accelerations recorded in the “far-field” are used as the basis of comparison between the two wall models. Amplification factors in the unreinforced backfill region are likely to be unaffected by the formation of the failure wedge and wall displacement. Therefore, to determine the effect of soil density on acceleration amplification, we compared the amplification factors determined in the unreinforced backfill region for KL4 and KL5.
Figure 4-61 shows that the 50% density KL5 model exhibits higher acceleration amplification in all excitation steps and in all the three backfill elevations compared to the KL4 model of 85% relative density. The looser backfill in the KL5 model will have a lower stiffness compared to the backfill in KL4. This is consistent with previous statements made regarding the relationship between backfill stiffness and acceleration amplification as the KL5 model with a lower stiffness exhibits greater acceleration amplification in all stages of excitation. The observations are in agreement with Kramer (1996) which states that at low accelerations, there is minimal non-linear soil behaviour which results in greater amplification for looser soils.

### 4.4.4.3 Influence of 3kPa surcharge

From Figure 4-62 and Figure 4-63, we observe that the surfaced models (KL9 – KL12) typically exhibit a lower acceleration amplification compared to their non-surfaced counterparts with a differential increase of amplification factors observed at the 0.4g excitation level. In the lower excitation levels, between 0.1g to 0.3g excitation, the differences in amplification factors were less than 0.05 (4%) but increases to a maximum difference of 0.1 (10%) at higher excitation levels. This observed difference is also greater for the top accelerometers placed at the height of 825mm with relatively little difference in the bottom accelerometers (225mm height).

The difference in amplification factors between surfaced and non-surfaced models is likely related to the behaviour of the surfage during model excitation. Due to its different stiffness, the surfage would have vibrated at a different frequency as the backfill. Although the surfage did not move during model excitation (observed movement of the surfage is due to deformation of the backfill beneath), its different frequency of motion and related inertial forces would have acted to reduce the dynamic forces applied to the backfill; acting as a damper. A reduction in dynamic forces experienced within the backfill would have resulted in a reduction of accelerations experienced. Similarly to the increase in vertical stresses within the backfill due to surfage, this damping effect of the surfage during dynamic excitation reduces with depth. As a result, the
surcharged models experiences less acceleration amplification compared to non-surcharged models with the difference in amplification reducing with increasing depth.

Figure 4-62: RMS acceleration amplification for the StagLH and LH075 wall models in the reinforced backfill (250mm from the FHR wall panel) : (a) Blue = Elevation of 825mm; (b) Green = Elevation of 525mm; (c) Red = Elevation of 225mm.

Figure 4-63: RMS acceleration amplification for the LH09 wall models in the reinforced backfill (250mm from the FHR wall panel) : (a) Blue = Elevation of 825mm; (b) Green = Elevation of 525mm; (c) Red = Elevation of 225mm.
4.4.5 Reinforcement Load development

Studies on reinforcement load development have been performed by several researchers such as Watanabe et al. (2003), El-Emam and Bathurst (2005, 2007) and Allen and Bathurst (2002). These case studies show that the formation of reinforcement loads are complex and are due to a combination of several interdependent parameters. Wall lateral displacement is assumed to increase the reinforcement loads due to large relative movement of the reinforcement layers but at the same time, the formation of a failure wedge due to a lateral displacement decreases the embedment length and subsequent reinforcement loads. Furthermore, increased rotational displacement of the wall with excitation leads to increased overturning forces being applied to the wall resulting in subsequent increases in reinforcement loads to maintain equilibrium.

4.4.5.1 Influence of reinforcement length and layout

As previously observed in Section 4.3, reinforcement load development differs with each reinforcement layer. Therefore, the influence of GRS wall parameters on reinforcement load development for each instrumented reinforcement layer is presented in the following figures (Figure 4-64).

The influence of an extended reinforcement layout on reinforcement loads is determined from comparisons of the StagLH and LH075 models. For the bottom R1 reinforcement layer, in both the non-surcharged and surcharged models, both LH075 and StagLH wall models exhibit similar reinforcement loads. This is likely related to the identical R1 reinforcement lengths in both models. Furthermore, reinforcement loads in the bottom reinforcement layer are not significantly affected by increased overturning forces that arise from greater wall rotational displacement.

For the top R5 reinforcement layer, relatively similar reinforcement load development is also observed despite longer reinforcement length present in the StagLH models. Due to its greater wall progression to failure, as rotational displacement in significant in all stages of excitation, the LH075 wall is more inclined compared to the StagLH model at the end of each excitation level. As a result, a greater overturning force is applied to the wall panel by the backfill. To maintain equilibrium of the wall, greater reinforcement loads are generated resulting in the higher than expected reinforcement loads for the LH075 model. Furthermore, with greater wall displacements, greater reinforcement loads are also generated from the increased pullout displacement of the reinforcement layer.

This situation is also evident in the reinforcement load development comparisons for the R3 reinforcement layer. Despite having similar R3 reinforcement lengths, comparing the non-surcharged models, the LH075 model exhibited significantly greater reinforcement loads in the middle reinforcement layer than the StagLH model due to greater wall displacements of the LH075 model at the end of each excitation level.
Figure 4-64: Reinforcement Load development for non-surcharged models in: (a) R1 reinforcement layer (Elevation 150mm); (b) R3 reinforcement layer (Elevation 450mm); (c) R5 reinforcement layer (Elevation 750mm).

To determine the influence of uniform reinforcement length, we compare the LH09 models and the LH075 models. In the non-surcharged model comparisons, the LH09 models generally show similar reinforcement loads in all stages of excitation compared to the shorter LH075 model for all three instrumented reinforcement layers except in the top R5 reinforcement layer where higher reinforcement loads are developed for the LH075 models. This observation is due to the greater wall displacement experienced by the LH075 models resulting in greater reinforcement loads developed at the end of each excitation level and the longer reinforcement length for the LH09 models which
resulted in greater reinforcement loads developed for a given wall displacement. For the R5 reinforcement layer, the influence of wall displacement on reinforcement load development is greater due to its high elevation (high overturning force), resulting in greater reinforcement loads developed for the LH075 model.

Note that higher reinforcement loads were observed in the top R5 reinforcement layer for the StagLH models compared to the LH09 wall models despite identical reinforcement lengths. This is related to the greater proportion of total reinforcement load taken up by the R5 reinforcement layer in the StagLH wall model. As the lower three reinforcement layers are of shorter length, a greater distribution of reinforcement load towards the longer top two reinforcement layers occurs.

4.4.5.2 Influence of soil density

Under static conditions, the higher density KL4 model of 85% relative density exhibited linear reinforcement load profile with 0.29 and 0.5 kN/m recorded in R5 and R1 reinforcement layer respectively (Figure 4-65). This differs with the near similar increases in reinforcement load for the 50% relative density models. In the KL4 model, greater interlock of the soil particles with the geogrid reinforcement is present due to the high density backfill, contributing positively to the geogrid-soil interaction of the reinforcement layer. Therefore, less wall movement is required to engage the geogrid layers. Comparisons of reinforcement load development with model excitation are made in Figure 4-66.

![Figure 4-65: Reinforcement Load development under static conditions for models without surcharge including KL4.](image-url)
In Figure 4-66, the KL4 higher density model is observed to have developed higher reinforcement loads in all excitation levels. This is because in the higher density model, greater confinement of the geogrid layer is present, leading to an increase in reinforcement pullout resistance (Alfaro et al., 2005). The higher density model also has larger destabilising forces acting on the GRS wall due to a greater backfill mass; this also contributes to the larger reinforcement loads experienced.

4.4.5.3 Influence of 3kPa surcharge

To determine the influence surcharging has on the reinforcement loads, surcharged and non-surcharged models of StagLH and LH075 are compared (Figure 4-67). Similar behaviour is observed for the LH09 models.

For all reinforcement layers, the surcharged wall models typically exhibited greater reinforcement loads compared to their non-surcharged counterparts at all excitation levels with a similar general load development observed. These observations indicate that the surcharge has minimal influence on the progression of reinforcement load development but does increase the magnitude of reinforcement load achieved.

The applied backfill surcharge results in an increase of the normal stresses within the soil backfill. This acts to increase the pullout resistance of the reinforcement layers, increasing the stability of the GRS wall model. However, separately, the surcharge also acts to increase the active earth pressure acting on the GRS wall model, thus increasing the destabilising forces acting on the model wall. An increase in pullout resistance and an increase in destabilising wall forces will both lead to an increase in reinforcement loads accumulated as observed in Figure 4-67.

An exception to this general trend is the reinforcement load development for the non-surcharged wall models in the middle R3 reinforcement layer which show significantly greater reinforcement loads in the LH075 model compared to the StagLH counterpart. For surcharged wall models, relatively similar reinforcement loads are observed for the StagLH and LH075 models. This is due to similar wall displacements experienced during excitation (Table 4-6). However, for the non-
surcharged wall models, significantly greater wall displacements were experienced for the KL5-LH075 model which would have led to greater overturning forces and increased pullout displacement of its reinforcement layers. This in turn would generate greater reinforcement loads within the model.

Table 4-6: Differences in wall top displacements between LH075 and StagLH, surcharged and non-surcharged models

<table>
<thead>
<tr>
<th>Excitation Levels</th>
<th>Accumulated wall top displacement (mm)</th>
<th>Surcharged models</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>LH075 Model – KL5</td>
<td>StagLH Model – KL8</td>
</tr>
<tr>
<td>Static</td>
<td>1.58</td>
<td>0.98</td>
</tr>
<tr>
<td>0.1</td>
<td>4.32</td>
<td>3.31</td>
</tr>
<tr>
<td>0.2</td>
<td>9.02</td>
<td>7.57</td>
</tr>
<tr>
<td>0.3</td>
<td>17.96</td>
<td>14.74</td>
</tr>
<tr>
<td>0.4</td>
<td>44.38</td>
<td>32.90</td>
</tr>
<tr>
<td>0.5</td>
<td>186.04</td>
<td>180.05</td>
</tr>
</tbody>
</table>

Figure 4-67: Comparisons for reinforcement load development between surcharged and non-surcharged models for StagLH and LH075 models: (a) Bottom reinforcement layer, R1 (150mm elevation), (b) Middle reinforcement layer, R3 (450mm elevation), (c) Top reinforcement layer, R5 (750mm elevation).
4.4.5.4 Influence of Failure Progression – Wall displacement

Previous case studies of GRS wall models have found a direct relationship between reinforcement loads and wall displacement. This was observed in El-Emam and Bathurst (2007), Watanabe et al. (2003) and is also observed in the present tests. To show the influence of wall displacement on reinforcement loads, reinforcement load – wall displacement plots for the non-surcharged LH075 and LH09 models are shown in Figure 4-68. From these figures, we hope to isolate the influence of wall failure progression (in the form of horizontal wall displacement) on reinforcement loads. Relatively similar reinforcement load – displacement relationships are observed between the wall models (similar progression of reinforcement load development with horizontal displacement), indicating good consistency in the experimental results.

Figure 4-68: Comparisons for reinforcement load development with horizontal reinforcement displacement between non-surcharged wall models: (a) R1 layer - 150mm elevation; (b) R3 layer - 450mm elevation; (c) R5 layer - 750mm elevation.
For the R1 reinforcement layer of 150mm elevation, the LH075 and StagLH models of identical R1 reinforcement lengths exhibit a lower reinforcement load for a given displacement compared to the LH09 models. This is likely related to the shorter R1 reinforcement length for the LH075 and StagLH models which would have resulted in a lower reinforcement pullout resistance. Furthermore, the R1 reinforcement layer does not contribute significantly to the wall overturning resistance and would not be strongly influenced by the rotational displacement of the wall. Therefore, for a given wall displacement, a lower reinforcement load is generated compared to the LH09 models.

Similar observations can be made about reinforcement load development for the R3 reinforcement layer as the LH09 models is observed to develop a higher reinforcement load with wall displacement. For a given displacement, the LH075 wall model exhibited higher than expected reinforcement loads in the R3 reinforcement layer compared to the reinforcement loads developed in the StagLH wall model which had an identical reinforcement length. This is due to the wall model’s greater wall inclination at the end of each excitation level which is not represented through the reinforcement layer’s horizontal displacement. This is shown in Figure 4-69 where at the 0.3g and 0.4g excitation level, the LH075 wall model is significantly greater inclined than the other wall models. The greater wall inclination experienced at the end of each excitation level would have resulted in greater overturning forces applied to the wall and thus greater reinforcement loads generated to maintain equilibrium.

Figure 4-69: Wall displacement of the non-surcharged models at: (a) 0.3g excitation level; (b) 0.4g excitation level.
In the top reinforcement layer, although the StagLH and the LH09 models have the same reinforcement length, the StagLH models is observed to develop larger reinforcement loads with wall displacement. This is likely due to a greater proportion of total reinforcement load taken up by the R5 reinforcement layer in the StagLH wall model as previously mentioned in Section 4.4.5.1. Furthermore, the top reinforcement layer might have prevented the failure plane from propagating up into the surface of the backfill from its points of origin, which was the ends of the shorter reinforcement layers below (Watanabe et al., 2003). Similar observations were made by Watanabe et al. (2003) regarding the reinforcement load development for extended reinforcement layers in a staggered reinforcement wall model. In addition to this, for both the LH075 and StagLH wall models, greater wall failure progression and thus greater wall inclination at the end of each excitation level resulted in larger overturning forces applied to the wall hence greater reinforcement load developed compared to the LH09 wall models.

4.5 Summary

In this chapter, the seismic performance of a series of twelve reduced-scaled GRS walls models with a full-height rigid facing was investigated. A testing summary of the wall models tested was first presented and the reliability of the experimental results from wall models tested discussed. Model excitation consisted of a staged sinusoidal excitation of 5Hz frequency and 10s duration for each excitation level. Acceleration amplitude started from 0.1g and increased at 0.1g increments until failure of the wall.

The performance of a representative wall model, KL5, a non-surcharged wall model reinforced with L/H = 0.75 and backfill relative density of 50% was initially focused on to present typical GRS wall behaviour and responses observed. Following that, the influence of individual wall parameters such as reinforcement length and layout, backfill density and surcharging on the seismic response of the wall models were then discussed. The key findings from this chapter are summarised as follows:

- A combination of shake-table amplitude controllability issues, wall seal issues and different instrumentation layout due to time constraints resulted in a suspect reliability of the results from the KL1 to the KL3 wall models. As a result, the results from these tests were not used in the subsequent analyses.
- In all excitation levels, the rotational displacement component was the dominant failure mode (incremental increase of 110mm at failure). Sliding displacement was only significant in the last two excitation levels of the wall model (at the critical acceleration point) with the greatest incremental increase observed at failure of 30mm.
- At failure, significant rotation of the reinforced soil block was observed with the development of a failure wedge and corresponding failure surfaces similar to that observed by Jackson (2010) and Watanabe et al. (2003).
- A bi-linear displacement acceleration curve was observed where beyond a certain threshold acceleration level, the rate of displacement of the wall panel increases rapidly indicating failure of the wall.
- Typically, the contribution of sliding to the total wall displacement initially decreases in the first two excitation levels and subsequently increases in the last two excitation levels. This is due to some displacement required for engagement of the geogrid in the initial excitation levels and the exceedance of the pullout resistance and frictional resistance of the wall toe in the larger excitation levels.
• Critical acceleration was defined as the acceleration level where a significant increase in sliding displacement of the wall panel is observed and was typically the excitation level before failure (0.4g for the StagLH and LH075 Dr = 50% wall models and 0.5g for the LH075 Dr=90% wall model and LH09 Dr=50% wall models).

• Gradual increase of acceleration amplification with increasing excitation levels were observed in the wall models (of factors up to 1.5, 1.45 and 1.42 achieved within the reinforced backfill, backfill interface region and unreinforced backfill region). This is due to the reduction of the natural frequency of the backfill towards the lower excitation frequency (a result of higher strains within the backfill) which lead to resonance effects within the backfill.

• At failure, a decrease of magnitude of acceleration amplification was observed in the reinforced backfill and backfill interface region (a reduction in the amplification factor by approximately 0.4). This resulted from high non-linear soil behaviour at failure (due to significant model deformation) preventing development of high accelerations in these regions.

• Acceleration amplification in the reinforced region was observed to be non-linear along the wall elevation with greater amplification observed at higher elevations.

• Static earth pressures recorded at the end of each excitation level within the model were relatively similar due to a combination of increased earth pressures due to tilting of the pressure cells with wall displacement and reduced earth pressure due to reduction of backfill density (due to backfill deformation) around the pressure cells.

• The pressure cell located at the wall toe (Press1) exhibited a steady development of residual earth pressures reaching a maximum earth pressure of 0.86σ₀' at failure for the representative wall model, KL5.

• Maximum seismic earth pressures within each excitation level are significantly less than theoretical earth pressures estimated assuming passive soil conditions. This is because of changes to the backfill properties (density) during excitation due to outwards displacement of the wall panel.

• Similar to static earth pressures, the pressure cell at the wall toe recorded the highest seismic earth pressure reaching 17kPa (or 1.5σ₀') at failure for the KL5 wall model of 50% backfill relative density.

• For reinforcement loads, the centreline load cell typically exhibits a pronounced magnitude of reinforcement load change at each excitation level which is in agreement with theoretical analysis of load distribution along the width of the reinforcement layer.

• The lowest reinforcement layer, R1 exhibits a gradual accumulation of reinforcement load up till the excitation level before failure, reaching a maximum reinforcement load per unit width of 1.15kN/m for the representative wall model. The decrease of accumulated reinforcement load at failure is due to the pullout failure of the R1 reinforcement layer resulting in the increased sliding displacement of the wall toe observed.

• An initial peaking of reinforcement load for the R3 reinforcement layer (0.7kN/m for the representative wall model) in the 0.2g excitation level is observed before increasing back to the peak reinforcement load at failure. The incremental decrease of the reinforcement load with increased reinforcement displacement indicates possible pullout failure or slips at the soil-geogrid interface; however, more studies should be performed to confirm this.
Gradual development of reinforcement load for the top R5 reinforcement layer is observed beyond 10mm of horizontal reinforcement displacement with a peak reinforcement load achieved at failure of 0.8kN/m. Greater incremental increase of reinforcement load developed at the larger excitation levels is likely due to increased wall top displacement.

An apparent load transfer from the R1 reinforcement layer to the R3 and R5 reinforcement layer at failure is observed and is likely due to pullout failure of the R1 reinforcement layer.

Within an excitation level, a slight phase difference between reinforcement load development and wall displacement is observed with the reinforcement load development reaching an earlier peak. This is due to the initial resistance of the reinforcement layer to outward displacement (increase in reinforcement loads recorded) before some slippage of the reinforcement is experienced in each loading cycle resulting in the residual displacement of the reinforcement recorded.

Regarding the influence of wall parameters such as reinforcement length and layout, backfill density and backfill surcharging on the GRS wall behaviour, the following conclusions were made in this chapter:

- The increase of backfill density (Dr=50% to Dr=85%) was found to have a greater effect on wall stability than an increase in reinforcement length by 20%. This was shown in the lower displacement-acceleration curves observed for the higher density wall model.
- As expected, longer total reinforcement lengths within the GRS wall models results in lower wall displacements. An increase in reinforcement length for the top two layers by 20% resulted in a reduction of wall top displacement by 10mm prior to failure. However, the model excitation increments of 0.1g was too coarse to differentiate between critical accelerations determined for the LH075 and StagLH models with both models achieving failure at the 0.5g excitation level.
- An increase in backfill density by 35% was sufficient to increase the critical acceleration determined for the wall models by 0.1g from (0.4g to 0.5g). However, note that the increase in critical acceleration would be dependent on the model excitation increments.
- The surcharge was observed to initially increase the wall stability due to greater normal stresses within the backfill. However, at larger excitation levels, the surcharge’s contribution to the wall destabilising forces out-weighed its contribution to wall stability and this is found to likely coincide with failure plane formation within the backfill.
- The effect of extended top reinforcement lengths in the StagLH models was to restrict the rotation of the wall under dynamic loading resulting in a greater sliding component contribution to wall displacement at failure. This is confirmed by the higher angle of rotation at failure of the wall panel for the LH075 models (9°) compared to the StagLH models (8.5°).
- Minimal differences in displacement component contribution (rotational or sliding components) are observed between models of different backfill densities. Further tests should be performed to confirm this observation.
- For the LH075 and StagLH wall models, the surcharge decreased the rotational component of wall displacement (increased sliding displacement) due to increased pullout resistance of the top two reinforcement layers.
- However, for the LH09 models, the surcharge would have had a larger contribution to the destabilising forces acting on the wall due to its larger failure wedge resulting in larger
overturning forces. As a result, greater rotational displacement was observed for the surcharged LH09 wall model.

- Critical accelerations did not differ between surcharged and non-surcharged wall models. However, the coarseness of the excitation level increments negatively affected identification of critical acceleration when differentiating between wall models with different reinforcement layouts.

- Lower amplification factors were determined for longer uniform reinforcement length models with the difference in amplification factors observed to be approximately 0.06 at the 0.4g excitation level at 825mm elevation. This is a result of reduced model deformation for the LH09 models resulting in a higher shear modulus and overall stiffness of the wall model.

- The shorter reinforcement models experiences greater backfill deformation and thus reduced stiffness of the structure. This results in greater amplification as the natural frequency of the deposit decreases and tends towards the excitation frequency of the model, increasing the resonance effect of the excitation.

- For models of lower backfill density, the looser backfill will have a lower stiffness compared to higher density models resulting in greater acceleration amplification. Amplification factors of 1.1 for the KL4 model compared to 1.25 for the KL5 model at 825mm elevation within unreinforced backfill were determined at failure.

- Due to its different frequency of motion and related inertial forces during excitation, the surcharge acted to reduce the dynamic forces within the backfill; acting as a damper. As a result, lower acceleration amplification was observed for surcharged models (a difference in amplification factors of approximately 0.1 was observed for the 825mm elevation).

- Typically, greater reinforcement loads are developed with greater wall displacements as greater pullout displacement of the reinforcement layers is experienced and a greater overturning force is applied to the wall. However, note that the bottom R1 reinforcement layer is not significantly affected by the overturning force applied to the wall due to its proximity to the wall toe.

- Greater reinforcement lengths also will result in greater reinforcement loads developed due to the greater pullout resistance of the reinforcement layer. This is observed in the greater reinforcement loads developed in the bottom R1 reinforcement layer for the LH09 models (1.3kN/m) compared to the LH075 models (1.0kN/m).

- For StagLH models, a greater distribution of reinforcement load towards the longer top two reinforcement layers is observed with the top R5 reinforcement layer developing higher reinforcement loads than the LH09 models despite identical reinforcement lengths (0.8kN/m compared to 0.6kN/m recorded at failure for the top reinforcement layer).

- For higher density models, a linear reinforcement load profile was achieved under static conditions indicating that less wall movement was required to engage the reinforcement layers due to greater soil-geogrid interaction within the model. Similarly, greater reinforcement loads are developed due to greater confinement of the reinforcement layer (1.9kN/m for the Dr=85% model compared to 1.1kN/m for the Dr=50% model recorded for the bottom R1 reinforcement layer at the excitation level prior to failure).

- The surcharge is observed to increase the magnitude of reinforcement load developed due to greater confinement of the reinforcement layer and increased destabilising wall forces. Minimal effect of the surcharge on reinforcement load progression is observed.
4.6 References


Chapter 5.0  PRE-FAILURE DEFORMATION OF THE GRS BACKFILL

5.1  Introduction
Post-earthquake case studies by Tatsuoka et al. (2008) and Ling et al. (1997) have shown that GRS walls have in the past performed well under seismic loading. The GRS walls in general demonstrated minimal to no damage during the 1994 Northridge and 1995 Kobe earthquakes, when conventional retaining walls failed. However, although failure of the GRS walls was not observed, there is a lack of understanding about the pre-failure performance of the GRS structure such as the progressive shear band development and failure wedge formation within the backfill. Hence, pre-failure deformations of the GRS walls are studied in this section.

Besides instrumenting the GRS model with accelerometers, LVDTs, load cells and earth pressure cells, deformation of the GRS wall models were externally monitored in the form of three high-speed cameras which focused on specific regions of the backfill. The high-speed images captured enabled the determination of shear and volumetric strain formation within the backfill throughout the testing stages.

Firstly, Section 5.2 presents a testing summary specifying the wall models selected to present GeoPIV results in this chapter. These wall models were chosen to best represent the effect of the varied GRS wall parameters such reinforcement length and backfill density on the strain development within the backfill. The angle of the final failure plane developed at the end of the bottom R1 reinforcement layer is also presented in this section. Section 5.3 elaborates on the GeoPIV technique used to determine soil shear and volumetric strains from the high-speed images captured of the backfill. A background discussion of the processes used and of the issues related to the current application of the process is presented in the section.

Section 5.4 presents the strain development of the chosen representative wall model, KL7 in the form of both residual (end of each excitation level) deformation plots and cyclic (within an excitation level) deformation plots. Strains are plotted as incremental and cumulative shear strain and volumetric strains. Also in this section, key features of strain development identified in residual deformation plots are discussed. In Section 5.5, strain development between different GRS wall models is analysed to determine the effect of each parameter on the pre-failure deformation of the backfill. Lastly, a summary of the chapter’s findings is presented in Section 5.6.

5.2  Testing Summary
Experimental results from wall models KL4 to KL12 (excluding KL9) were selected to study the pre-failure deformation mechanisms within the backfill in this section. The wall models chosen for GeoPIV analysis were unaffected by experimental shortcomings in both model construction and image capture and hence allow for a high level of confidence when assessing each deformation response. Wall models KL1 to KL3 were not used in the GeoPIV analysis as the tests were affected by reliability issues as mentioned in Section 4.2. Furthermore, low quality high-speed images were captured during these tests due to insufficient camera resolution and the use of fewer high-speed cameras in the first stage of experimentation. Wall models whose GeoPIV results were chosen to be presented in the body of this thesis are highlighted in green in Table 5-1.
GeoPIV results from all wall models are not presented in the main body of the text due to similar differences in strain development between wall models of a varied GRS parameter. For example, the differences in strain development between the surcharged models and non-surcharged models are similar regardless of the reinforcement layout; as a result, the GeoPIV results for the KL10 wall model is presented (for comparison to the KL7 wall model). GeoPIV results for all valid tests (including those which are not reported in this chapter) are further presented in Appendix D.

Table 5-1: Table of valid tests selected for GeoPIV analysis

<table>
<thead>
<tr>
<th>Wall Model</th>
<th>Reinforcement Layout, L/H</th>
<th>Density (%)</th>
<th>Surcharge (kPa)</th>
<th>Acceleration at failure (g)</th>
<th>Test Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>KL4</td>
<td>0.75</td>
<td>85</td>
<td>-</td>
<td>0.6</td>
<td>Results are not presented for ROI (i)¹: Sunlight reflection and incorrect exposure</td>
</tr>
<tr>
<td>KL5</td>
<td>0.75</td>
<td>50</td>
<td>-</td>
<td>0.5</td>
<td>-</td>
</tr>
<tr>
<td>KL6</td>
<td>0.9</td>
<td>50</td>
<td>-</td>
<td>0.6</td>
<td>-</td>
</tr>
<tr>
<td>KL7</td>
<td>0.9</td>
<td>50</td>
<td>-</td>
<td>0.6</td>
<td>Representative Wall Model</td>
</tr>
<tr>
<td>KL8</td>
<td>Top two layers @ 0.9, Rest @ 0.75</td>
<td>50</td>
<td>-</td>
<td>0.5</td>
<td>-</td>
</tr>
<tr>
<td>KL10</td>
<td>0.9</td>
<td>50</td>
<td>3</td>
<td>0.6</td>
<td>-</td>
</tr>
<tr>
<td>KL11</td>
<td>0.75</td>
<td>50</td>
<td>3</td>
<td>0.5</td>
<td>-</td>
</tr>
<tr>
<td>KL12</td>
<td>Top two layers @ 0.9, Rest @ 0.75</td>
<td>50</td>
<td>3</td>
<td>0.5</td>
<td>-</td>
</tr>
</tbody>
</table>

¹ Location of Region of Interest (ROI) (i) is presented in Section 5.3.3

5.2.1 Interpretation of deformation from global images

During excitation, the global deformation of each wall model was recorded through a series of images taken of the entire backfill at the end of each excitation level, termed global images. Global images for the KL5 wall model have been previously presented in Section 4.3.1. Due to issues caused by black sand lines on subsequent GeoPIV analysis within the wall model previously mentioned in Chapter 3, vertical black sand lines were not constructed continuously throughout the backfill. Unfortunately, because of this, we are unable to accurately determine the position and angle of the shear surfaces developed within the backfill during excitation from the global images leading to an incomplete picture of development of shear planes within the backfill during excitation of the wall model from global images. However, note that GeoPIV analyses of each region of interest supplements the incomplete information of backfill deformation from global images and will be later discussed in Section 5.4.

Only the angle of the final failure plane within the unreinforced backfill, which becomes prominent at the failure excitation level, can be determined from the global images. The characteristics of the final failure plane is consistent for all wall models and typically extends from the surface of the backfill to the end of the R1 reinforcement layer at an inclined angle. This is shown in Figure 5-1 where the failure planes determined from the global images for the representative KL7 wall model is outlined with black dashed lines. The angles of the final failure plane in the unreinforced portion of the backfill determined from global images of all valid tests are presented in Table 5-2.
For the uniform reinforcement wall models, the application of surcharge does not appear to affect the failure plane angle as both types of wall models show relatively similar values. Between the LH075 and LH09 models, the LH09 models exhibited a shallower failure plane compared to its shorter uniform reinforcement counterpart. This has been also observed by Watanabe et al. (2003) in 1-g shake-table wall model studies and it likely due to the higher stability of the LH09 wall model. Similarly, the higher density KL4 model also displays a shallower failure angle compared to the lower density KL5 model. For the staggered reinforcement models, the effect of the surcharge appears to have increased the angle of the developed failure plane with the non-surcharged model displaying a relatively shallow failure plane, while the opposite is observed for the surcharged model.

Note that due to the relative small data set, further research in the form of more wall models tested or a compilation of previous wall model studies will have to be done to verify these findings.
5.3 GeoPIV Technique

During each excitation level, high-speed images of each region of interest were captured and stored. Image data was then analysed using GeoPIV to determine the backfill displacement and strain fields within these regions. From the GeoPIV analysis, essential information regarding the backfill deformation can be obtained such as the location and angle of failure planes, the timing of formation (as number of cycles) and the magnitude of local strains.

5.3.1 Background of GeoPIV

Particle image velocimetry (PIV) technique was originally used in experimental fluid dynamics applications (Adrian, 1991) where the fluid was seeded with tracer particles which were then tracked to enable a velocity field to be determined. Typically, PIV involves the use of high-speed cameras to capture image information at a predetermined rate for analysis with the PIV software. In general, PIV enables image texture, through different colourations of soil particles, to be identified and their displacement measured from successive high-speed images over a known time difference. The PIV software used in the present analyses was specifically developed for geotechnical applications by White et al. (2003) and termed GeoPIV (Geotechnical PIV).

In GeoPIV, the initial image is first divided into a mesh of patches (size set to a certain number of pixels) with the texture (different colouration of soil particles) recorded. To find the displacement of a singular test patch between the subsequent images, a search patch, which extends a specified distance beyond the test patch, is extracted from the second image. The cross-correlation of the test patch and the search patch is evaluated and the resulting normalised correlation plane will indicate the ‘degree of match’ between the test and search patch. The highest peak in the normalised correlation plane indicates the displacement vector of the test patch (White et al., 2003). This operation is repeated for the entire mesh of patches, thus producing the displacement field between the images. The vector field determined from GeoPIV analysis is calibrated from “image-space” (in pixels) to “object-space” (in mm) by correlation with a grid of fixed points (“reference points”) on the surface of the transparent Perspex window built into the box.

The image texture must be sufficient for the GeoPIV algorithm to recognize the discrete patches accurately, or external particles will have to be “seeded” into the specimen (Bolinder, 1999). The process of “seeding” involves the addition of more visible particles to the soil to alter the contrast of the material and thus increase the image texture, allowing for easy recognition by the GeoPIV software during image analysis. In Jackson (2010), “seeding” of small pebbles into the sand layered closest to the Perspex sidewall was trialled but deemed unnecessary as the Albany sand and lighting conditions enabled sufficient texture of the backfill to be captured by the high-speed cameras.

The general GeoPIV processes can be summarised in the illustration shown in Figure 5-2 (White & Take, 2002).
5.3.2 Discussion of GeoPIV process

The performance of the GeoPIV process is dependent on three factors: (i) the precision, (ii) the accuracy and (iii) the number of measurement points that can be established in an image (White et al., 2003). The precision of GeoPIV is defined by the difference between the true movement of the patches and that recorded by through image-capture and analysis. The accuracy of the GeoPIV process involves the transformation of the vector field generated in “image-space” (in pixels) to “object-space” (in mm).

5.3.2.1 GeoPIV precision

A series of experiments were conducted by White et al. (2003) to assess the precision of the GeoPIV process. The experiments involved the controlled rigid-body movement of a planar body below a fixed camera. Images used in the GeoPIV analysis were a mixture of computer-simulated random images and physical experiments of Dog’s Bay sand and kaolin clay dusted with dyed sand. Experimental results showed that precision is a strong function of patch size (with larger patches having greater precision) and a weak function of image content (patches of low texture are more difficult to locate precisely).

From their experiments, White et al. (2003) derived an upper bound curve on the precision error, $\rho_{\text{pixel}}$, which can be expressed as ($L$ is the patch size):

$$\rho_{\text{pixel}} = \frac{0.6}{L} + \frac{150000}{L^8}$$  \hspace{1cm} (5-1)

The equation allows for a conservative estimate of the random errors present in the PIV data. The precision error can be expressed as a fraction of the ROI by dividing the error, $\rho_{\text{pixel}}$, by the image width in pixels.
5.3.2.2 GeoPIV accuracy

The accuracy of the GeoPIV process can be broken down into the positional accuracy and the movement accuracy of the system (White et al., 2003).

The positional accuracy of the GeoPIV process is dependent on the method used to establish the object-space location of the reference points; termed the calibration process. To accurately convert between pixels and object coordinates, photogrammetric transformation from image-space (uv-space) to object-space (xy-space) via the usage of a Mylar sheet is performed. A Mylar sheet is a sheet with a grid of dots of which their xy-space coordinates are accurately known. The object-space coordinates for the reference points are thus derived from the accurately known Mylar sheet points.

The movement accuracy of the system is derived from the errors of the PIV operations used to track the soil in image space and track the reference points from which the image-space and object-space transformation parameters are derived. In other words, the movement accuracy of GeoPIV process is dependent on the precision of tracking each soil patch and each reference point between successive images. Geometric summation of both errors gives an estimate of the GeoPIV movement accuracy (White et al., 2003).

White et al. (2003) stated that the number of measurement points that can be established in an image is a function of the PIV patch size. As a result, the number of measurement points can be increased by using smaller patch sizes, at the cost of reduced precision. A greater number of measurement points will reveal greater detail from the GeoPIV analysis, especially in areas of high strain gradient. This was confirmed by Jackson (2010) who chose a 32 by 32 pixel mesh as a compromise between the detail revealed by the GeoPIV process and GeoPIV precision.

However, White et al. (2003) and Jackson (2010) performed their GeoPIV analysis by creating a mesh of non-overlapping patches, with the gap between patches equal to the size of each patch. In a
different approach, GeoPIV analysis performed in Bowman et al. (2011) used 32 pixel patches that overlapped at 8 pixel intervals. Overlapping of patches was also previously explored by Lesniewska and Wood (2009) in their studies of internal stress and deformation fields of a sample. By overlapping the patches, the number of measurement points within a single image can be increased so the number of measurement points in the system is determined by the spacing between the patches. This is shown in Figure 5-4 where a patch spacing of 8 pixels instead of 48 pixels increases the number of measurement points significantly. As a result, the overlapping of patches increased the precision of the strain field developed, resulting in greater detailing of the regions of high-strain gradient in the analysis.

Figure 5-4: Mesh of soil patches and corresponding displacement vector plots from GeoPIV analysis of KL7 for ROI (ii): (a) 48 pixel patches with patch spacing of 48 pixels; (b) 48 pixel patches with patch spacing of 8 pixels.
5.3.3 ROI dimensions in the backfill in current application

Three independent regions of interest (ROIs) were focused on in the GeoPIV analysis. Locations of each ROI for a representative model and its dimensions are shown in Figure 5-5. Each ROI’s dimensions were kept constant for all wall models with only the locations of each ROI altered for the different reinforcement layouts.

To validate the ROI dimensions chosen for GeoPIV analysis, Jackson (2010) investigated the ability of the GeoPIV analysis to record displacement over the first half cycle of a 0.1g shaking motion, generated by a shake-table displacement of 1mm. This calibration exercise showed that the images captured had sufficient texture for soil tracking and thus confirming the suitability of the ROI dimensions chosen for GeoPIV analysis. Therefore, for the current application, to ensure the suitability of the ROI dimensions used in current tests, similar pixel-to-mm ratios that were achieved by Jackson (2010) were aimed for when sizing the three regions of interest. The pixel-to-mm ratios achieved in the current set of tests are shown in Table 5-3.

For ROI (i), a lower pixel-to-mm ratio of 2.7 was achieved compared Jackson (2010). However, due to the Phantom Miro’s advanced features that resulted in a significantly higher detail and clarity in the high-speed images captured. As a result, the slight difference in pixel-to-mm ratio for ROI (i) was not deemed critical as the greater detailing from the Phantom Miro camera would offset this. Note that for ROI (iii), a higher pixel-to-mm ratio was achieved compared to Jackson (2010).

![Figure 5-5: High-speed Camera ROI layout for L/H = 0.75 wall models.](image)

<table>
<thead>
<tr>
<th>Camera</th>
<th>Camera Resolution (pixels), W x H</th>
<th>ROI dimension (mm), W x H</th>
<th>Pixel/mm ratio, W x H</th>
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<tr>
<td>ROI (i)</td>
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<tr>
<td>ROI (ii)</td>
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<tr>
<td>Jackson (2010)</td>
<td>MotionPro X3 1024 x 1280</td>
<td>300 x 400</td>
<td>3.4 x 3.2</td>
</tr>
</tbody>
</table>
5.3.4 Discussion of GeoPIV for the current application

In this section, a discussion of key GeoPIV analysis parameters is presented to determine optimum values to be used in the subsequent GeoPIV analysis of all wall models. Additionally, some experimental issues regarding GeoPIV analysis are also discussed.

5.3.4.1 Determination of optimum patch spacing

To determine the effect of patch spacing on the GeoPIV strain analysis, four different patch spacing were trialled on the backfill deformation of ROI (ii) in the representative wall model KL7. Figure 5-6 shows the accumulated residual shear strain plots developed after the 0.4g excitation level.

From Figure 5-6, we observe that the different patch spacing does not appear to influence the location of each individual strain band but instead influences the detailing of the strain band formation. GeoPIV analysis performed with a patch spacing of 4 or 8 pixels produced strain plots with clear and distinct strain bands. Conversely, blurred and low-detailed strain bands were developed when a patch spacing of 24 or 48 pixels were used. This is in agreement with previous statements made in Section 5.3.2.3 that relate the patch spacing to the level of detailing achieved in regions of high-strain gradient.

The increased detailing due to smaller patch spacing resulted in each individual strain band appearing narrower with strains highly concentrated along the strain bands. In other words, each individual shear band becomes more distinct from its counterparts as the patch spacing is decreased. As a result, presentation of a greater degree of strain concentration along each individual strain band is observed when patch spacing is decreased. This is indicated by higher shear strain levels achieved along the inclined strain bands when a 4 pixel patch spacing was used (Figure 5-6(a)) compared to those achieved with a 8 pixel patch spacing (Figure 5-6(b)). Note that between Figure 5-6(a) & (b), the number and position of the strain bands is the same but they are more distinct when using a patch spacing of 4 pixels (thus recording greater strain levels as mentioned previously).

Differences in the level of detailing achieved in GeoPIV analysis performed by using 4 pixels patch spacing compared to 8 pixels are observed to be minor. However, the GeoPIV analysis takes double the computational effort to produce a mesh of patches with 4 pixel patch spacing compared to a mesh of 8 pixel patch spacing, while there is little actual difference in the shear band patterns observed between the two spacings. Therefore, to achieve the balance between computational effort and level of detailing from the GeoPIV analysis, an optimum patch spacing of 8 pixels was chosen in this research.

Figure 5-6: Accumulated shear strain plots (in %) of KL7 for ROI (ii) after the 0.4g excitation level produced from GeoPIV analysis of 48 pixel patches with different patch spacing: (a) 4 pixels; (b) 8 pixels; (c) 24 pixels and (d) 48 pixels.
5.3.4.2 Determination of optimum search zone

In the event of high backfill deformation, the search-zone used in the GeoPIV will have to be large enough to encompass the entire strain field so that the initial soil patch that is being tracked will be within the search zone. As the application of GeoPIV analysis for this research involves the dynamic movement of the soil, the search zone chosen also has to cover the physical movement of the shake-table over its displacement range to enable tracking of the initial soil patches.

By increasing the size of the search-zone in the GeoPIV analysis, we can increase the likelihood of tracking a soil patch in the event of high backfill deformation. However, the effect of increasing the search area can have an adverse effect on the precision of the GeoPIV analysis. If the search zone is large, the GeoPIV analysis is more likely to track another soil patch that is unrelated to the initial soil patch rather than the “true” patch (which may give a lower correlation due to the image distortion), leading to wild vectors being formed.

This agrees well with observations of GeoPIV analysis that show a concentration of wild vectors developed at locations of high strain development (Figure 5-7). The propensity for the analysis to track a different soil patch than the original initial soil patch increases with a larger search zone; an increase in imprecision of the GeoPIV analysis (higher probability of tracking an incorrect soil patch that has a high correlation to the initial soil patch). This is confirmed in observations in Figure 5-7 where the number of wild vectors in the displacement vector plots increases when larger search zones are used. Therefore, it is imperative that a balance is achieved so that the search-zone chosen is high enough to enable successful tracking and quantification of soil patch movement, but low enough to reduce the imprecision in the GeoPIV analysis in tracking soil patches.

After removing the wild vectors from the displacement vector plots, similar strain plots are produced from the GeoPIV analysis despite the different search-zones used. Note that the number of wild vectors removed is proportionate to the number of wild vectors developed in the strain plots (i.e. larger search zone, greater number of wild vectors, greater number of wild vectors removed). A slightly greater degree of strain accumulation is presented in strain plots produced from GeoPIV analyses that used larger search-zones. This is associated with the increased amount of wild vectors produced by the GeoPIV analysis when using high search zones. It is noted that the variation of search-zones in the GeoPIV analysis appears to have no effect on the location or the clarity of strain planes produced (Figure 5-8).

In conclusion, analyses carried out with a larger search-zone typically result in a greater number of wild vectors presented from the GeoPIV analysis. This is due to reduced precision of the GeoPIV analysis when using a high search-zone as the likelihood of the GeoPIV analysis tracked an unrelated soil patch to the initial soil patch increases. This is evidenced by the increase in wild vectors developed in high search-zone analyses shown in Figure 5-7. Variation of search-zones appears to have no effect on the location or clarity of the strain planes. A search-zone size of 150 pixels was chosen as the optimum search-zone size as it requires the least computational effort without a loss in precision but also enabled the high precision in the GeoPIV analysis in all cases.
5.3.4.3 Determination of the optimum patch size

From Section 5.3.2, it is understood that a compromise is needed in determining an optimum patch size with two conflicting interests to be balanced: Larger patches that provide improved precision (less wild vectors) and smaller patches that provide greater detail in regions of high strains. In addition to that, analysis in Bowman et al. (2011) showed by overlapping patches, greater detail can be achieved from the GeoPIV analysis.

To determine the optimum patch size to be used, four different patch sizes were trialled in the GeoPIV analysis of each ROI for the KL7 model: (i) 32; (ii) 48; (iii) 64 and (iv) 96 pixel patches (Figure 5-9 to Figure 5-11). The optimum patch spacing of 8 pixels is used in all four types of GeoPIV analyses. The total maximum shear strain determined from GeoPIV analysis is used as the basis of comparison to determine the optimum patch size.
Firstly, a reduction of detail in strain band formation is clear when comparing strain plots produced with 96 pixels patches and 32 pixel patches. For example, for ROI (ii), Figure 5-10(a) with 32 pixel patches shows a distinct ‘fish-net’ or lattice formation of shear bands. However, in Figure 5-10(d), this is no longer clear, with strain bands appearing to form parallel with one another. Similar observations can be made from strain plots produced for ROI (i) where a reduction of the number of individual strain bands is observed.

A reduction in detail when using larger patch sizes observed in Figure 5-10 is due to the “smearing” of the displacement field in the area of high strain gradient (White and Take, 2002). Although the overlapping of patches enables a greater level of detail to be observed, if the patch size is too big such that it covers the entire area of high strain formation, a loss of detail still occurs as the high strains developed are lost within the large soil patch (“smearing”). Therefore, although small patch spacing enables greater detail to be seen, a soil patch of reasonable size must be used to prevent the loss of detail in areas of high strain.

Conversely, GeoPIV analyses performed for ROI (iii) show that greater clarity of strain band formation was achieved when the soil patches were increased in size (Figure 5-11). When soil patches of 32 pixels were used, large random patches of high strain were observed in the strain plots.
which distorted the ability of the observer to pick out the strain bands. These random patches of high strain are due to wild vectors that were formed through the GeoPIV analysis. By increasing the soil patch size, the GeoPIV analysis precision can be improved and thus reducing the number of wild vectors formed. Figure 5-11 show that the increase of soil patch size from 32 pixels to 48 pixels was sufficient to achieve clarity in the strain band formation for ROI (iii). Note that the increase of wild vectors (random high strain patches in strain plots) due to the use of a smaller soil patch size of 32 pixels were also observed in ROI (i) and (ii) albeit to a lesser degree.

In conclusion, from results shown in Figure 5-9 to Figure 5-11, the optimum patch size of 48 pixels for the GeoPIV analysis was decided upon. Although, the patch size of 32 pixels provides slightly greater detail, far greater amounts of wild vectors were produced in its displacement vector plots. Comparatively, the 64 pixel and 96 pixel patch sizes provide lesser detail in areas of high strain compared to analyses performed with patch sizes of 48 pixels.

5.3.4.4 Inability to track soil patches due to large deformations

For ROI (i), an unmanageable amount of wild vectors was encountered in the GeoPIV analysis for the final two excitation levels resulting in nonsensical shear strain plots developed (Figure 5-12 & Figure 5-13). GeoPIV analysis performed in the ROI (ii) and ROI (iii) regions also encountered excessive amount of wild vectors but in the final excitation level only. This is shown in displacement vector plots and strain plots for ROI (ii) in Figure 5-14 and Figure 5-15. In both cases, for all regions, the development of an excessive amount of wild vectors at particular excitation levels is due to the excessive incremental deformation of the backfill within the region of interest.

![Figure 5-12: Displacement vector plots for KL7 for ROI(i) with 48 by 8 pixels patches: (a) After 0.4g excitation level; (b) After 0.5g excitation level; (c) At failure (0.6g excitation level).](image)

![Figure 5-13: Accumulated shear strain plots (in %) of KL7 for ROI(i) with 48 by 8 pixels patches: (a) After 0.4g excitation level; (b) After 0.5g excitation level; (c) At failure (0.6g excitation level).](image)
Figure 5-14: Displacement vector plots for KL7 for ROI(ii) with 48 by 8 pixels patches: (a) After 0.4g excitation level; (b) After 0.5g excitation level; (c) At failure (0.6g excitation level).

Figure 5-15: Accumulated shear strain plots (in %) for KL7 for ROI(ii) with 48 by 8 pixels patches: (a) After 0.4g excitation level; (b) After 0.5g excitation level; (c) At failure (0.6g excitation level).

This difference between ROI (i) and ROI (ii & iii) is due to the different magnitudes of backfill deformation experienced in each region of interest. In the excitation level before failure, high settlements were observed in the backfill, particularly the backfill interface region. This was also observed by Jackson (2010) and Sabermahani et al. (2009) in their model tests. These high settlements are likely due to the formation of the failure wedge that typically extends from the bottom R1 reinforcement layer to the top of the backfill surface, encompassing the entire ROI (i). During the formation of the failure wedge, the soil within ROI (i) translates rapidly, moving out of the window of reference points and out of the image. As a result, the GeoPIV analysis was unable to track the patches reliably during the last two excitation levels.

Furthermore, Jackson (2010) stated that the first shear plane developed is typically the shallowest shear plane, with an increase in excitation level either propagating the existing shear plane or generating a deeper shear plane. This observation indicates that for the backfill interface, the top portion of the backfill (ROI (ii)) generally experiences greater deformation compared to the lower regions of the backfill (ROI (iii)) at any given excitation level.

As ROI (ii) experiences less backfill deformation compared to ROI (i), the GeoPIV analysis is able to cope with the magnitude of deformation experienced at the excitation level before failure, thus producing reasonably clear strain plots which have a manageable amount of wild vectors. Conversely, for ROI (i), the GeoPIV analyses from the last two excitation levels of each wall model are not used due to the high number of wild vectors brought about by the large incremental deformation experienced in ROI (i) during the two excitation levels.
5.3.4.5 Influence of sunlight reflection on GeoPIV analysis

In the testing of the KL4 wall model, a detrimental effect of sunlight reflection on the GeoPIV analysis was observed. This is related to the texture of each soil patch, defined as the combination of different-coloured grains and the light-shadow formed between adjacent grains when illuminated (White et al. 2003).

White et al. (2003) stated that the texture of each individual soil patch detected by GeoPIV analysis is defined as the combination of different-coloured grains and the light-shadow formed between adjacent grains when illuminated. As the sunlight-lit portion of ROI (i) is unevenly illuminated compared to the rest of the ROI, the GeoPIV analysis is unable to accurately track the soil patches within the illuminated portion of the ROI. To mitigate this issue, two dust screens were used to shield subsequent GRS wall models tested onwards from excessive exposure to sunlight.

5.4 Strain development for representative GRS wall model

In this section, the pre-failure deformation of a representative wall model is first focused on to provide a platform of understanding of the backfill deformation of a typical GRS wall model in this research. The non-surcharged KL7 wall model which has a uniform reinforcement layout with L/H of 0.9 and a backfill relative density of 50% is used as the representative wall model. Note that the KL7 GRS wall model failed at the 0.6g excitation level.

In this section, the pre-failure deformations of the backfill in each of the three regions of interest are presented along with key observations and analyses made. Pre-failure deformation of the backfill can be broken down into two sections: (1) Residual deformation determined at the end of each excitation level throughout the testing sequence and (2) Cyclic deformation within an excitation level of interest. From the GeoPIV analysis, pre-failure deformation of the backfill can be presented through cumulative or incremental shear and volumetric strain plots.

Note that along a shearing surface, the soil typically undergoes contraction before dilating with increasing shear strain. Bowman et al. (2011) also reveals that prior to full development of a failure surface, alternating zones of contraction and dilation typically develop along the shear band.

5.4.1 Residual deformation plots

The residual deformation is defined as the deformation that occurred between the start of the test and the end of each excitation level. Images are first selected from the start of the test and at the end of each excitation level and then compared and analysed through the GeoPIV process. As previously stated in Section 5.3.4.4, strains developed in the last two excitation levels (0.5g and 0.6g) for ROI (i) and the last excitation level (0.6g) for ROI (ii) and (iii) were too large to be resolved by the GeoPIV analysis and thus were not displayed in the subsequent strain plots.

To understand the pre-failure backfill deformation of the GRS wall, both the cumulative and incremental shear and volumetric strain plots of all three regions of interest are presented in Figure 5-16 to Figure 5-21. Although both cumulative and incremental strain plots are based on the same residual images captured, the incremental strain plots exhibit a smaller strain range as they show strain developed between each excitation level. This enables key features that might not be picked up in the cumulative strain plots to be shown, thus providing a better understanding of residual strain development. The reinforcement layers are identified in the first strain plot for each region in the following figures.
In the following strain plots, for ease of comparison between the three regions, the same strain levels are used for all three regions. A maximum shear strain level of 20% was selected for the following residual deformation plots as a shear strain level of 20% is deemed to be the critical shear strain level that would result in distinct alternating zones of contraction and dilation along the shear plane (indicating that the soil has sheared to critical state). A corresponding volumetric strain level of +/- 10% was selected with positive volumetric strain indicating contraction of the soil. Furthermore, to ensure strain detail in the surrounding backfill is maintained (oversaturation of the strain details is avoided), the maximum strain levels are limited; although strain achieved along the shear band would be greater than the pre-determined maximum range.

Figure 5-16: (A) Cumulative and (B) Incremental shear strain plots (in %) of KL7 for ROI(i) at the completion of: (a) 0.1g; (b) 0.2g; (c) 0.3g and (d) 0.4g excitation level.

Figure 5-17: (A) Cumulative and (B) Incremental volumetric strain plots (in %) of KL7 for ROI(i) at the completion of: (a) 0.1g; (b) 0.2g; (c) 0.3g and (d) 0.4g excitation level.
Figure 5-18: (A) Cumulative and (B) Incremental shear strain plots (in %) of KL7 for ROI(ii) at the completion of: (a) 0.1g; (b) 0.2g; (c) 0.3g; (d) 0.4g and (e) 0.5g excitation level.

Figure 5-19: (A) Cumulative and (B) Incremental volumetric strain plots (in %) of KL7 for ROI(iii) at the completion of: (a) 0.1g; (b) 0.2g; (c) 0.3g; (d) 0.4g and (e) 0.5g excitation level.
Figure 5-20: (A) Cumulative and (B) Incremental shear strain plots (in %) of KL7 for ROI(iii) at the completion of: (a) 0.1g; (b) 0.2g; (c) 0.3g; (d) 0.4g and (e) 0.5g excitation level.

Figure 5-21: (A) Cumulative and (B) Incremental volumetric strain plots (in %) of KL7 for ROI(iii) at the completion of: (a) 0.1g; (b) 0.2g; (c) 0.3g; (d) 0.4g and (e) 0.5g excitation level.
5.4.1.1 **Key Observations based on Residual Plots**

Most of the observations made in this section will be based on the cumulative strain plots generated by GeoPIV with the incremental strain plots providing supporting information. As shear strains typically develop hand in hand with volumetric strains, observations of shear strain development are generally accompanied by corresponding volumetric strain development.

Firstly, accumulation of strain along each reinforcement length is observed in ROI (i) and ROI (ii), with a shear strain level of 20% or greater and a contractive strain level of 10% or greater achieved within the 0.1g excitation level (Figure 5-16 to Figure 5-19). Subsequent excitation levels brought about reduced strain development as observed in the incremental strain plots with strain development is observed to be the greatest in the initial 0.1g excitation level along the reinforcement length. At all excitation levels, mostly contractive or compressive strains were developed, evidenced by the highly contractive strain band observed along the reinforcement lengths in the cumulative strain plots. This is different from typical shear surfaces within the soil which generally exhibited alternating zones of contraction and dilation along the shear surface. The development of these strain bands is due to the pull-out displacement of the reinforcement layers under excitation and is also observed in results from GeoPIV analysis of images from Jackson (2010). Further analysis of these strain bands is shown in Section 5.4.2.2.

In relation to the above, for the 0.1g excitation level, although high shear strains (20% or greater) developed at the end portion of the R1 reinforcement layer, similar levels of strain development were not observed at the front end of the R1 reinforcement layer (wall toe) in Figure 5-20 (for ROI (iii)). Similar shear strains were only achieved at the 0.2g excitation level along the R1 reinforcement layer in ROI (iii) which also corresponded to contractive strains of 10% or greater. Note that the commencement of strain development only 140mm from the GRS wall toe is due to the missing portion of the geogrid reinforcement layer required for load cells installed at the front of the reinforcement layer. Strain accumulated at this location is due to the steel geogrid plates as the reinforcement layer undergoes pull-out displacement under excitation. It is assumed that the strain accumulated at this point would have been distributed along the excluded reinforcement length that would have extended between 0mm and 140mm.

In the backfill interface region, the strain accumulation is observed to be concentrated in the top three reinforcement layers, R5 to R3 during the first two excitation levels (0.1g to 0.2g). This is evidenced by a greater number of radiating strain bands from each reinforcement end and the development of an inclined preliminary shear band of 20% shear strain or greater after the 0.2g excitation level for each of the top three reinforcement layers. The radiating strain bands observed for the top three reinforcement layers accumulated shear strains of 8% and contractive strains of 4%. Note that in the ROI (i), a portion of the strain bands observed is an extension of the preliminary shear band developed at the R3 reinforcement layer. For the bottom two reinforcement layers (R1 and R2), the strain bands developed shear strain levels of 4% and contractive strains of 2-3%. In these initial stages of excitation, mostly contractive strains were developed. As observed in ROI (ii), these radiating strain bands formed a lattice-like structure as they intersected in the unreinforced backfill. However, in ROI (i), no lattice-like structure was observed with the strain bands developing parallel to the preliminary shear band. At the wall toe, for ROI (iii), no significant strains were developed in the initial two excitation levels except for the strain accumulation along the R1 reinforcement layer.
The preliminary shear bands developed at the reinforcement ends typically extend upwards towards the surface of the backfill at an inclined angle. A cumulative shear strain level of 20% or greater observed corresponded to alternating zones of contractive and dilative strains of +/- 10% or greater along the length of the preliminary shear band, indicating a potential failure surface (Bowman et al., 2011). Further development of the preliminary shear band at the R3 and R4 reinforcement layer appears to be halted between the 0.3g and 0.4g excitation levels evidenced by the unchanged length of the strain band up to the 0.4g excitation level and low strain development in the incremental strain plots. Although parallel radiating shear bands of lower shear strain (< 10% shear strain) from the preliminary shear band developed at these excitation levels, merging of these radiating shear bands to form a single dominant shearing surface (similar to the preliminary shear band) was not observed. Note that in ROI (i), parallel radiating shear bands corresponding to dashed bands of dilative strains (-5 to -10% volumetric strain) developed between patches of contractive strains of 10%, most obvious in the 0.4g excitation level. This is in agreement with observations from Bowman et al. (2011) where contraction appears as a relatively more diffuse process compared to dilation that tends to develop in thin bands.

For the 0.3g and 0.4g excitation levels, a greater concentration of strain development is observed for the bottom two reinforcement layers (R1 and R2) in the backfill interface compared to the top three reinforcement layers. This is indicated by a greater incremental increase in shear strain along the preliminary shear bands observed for the R1 and R2 reinforcement layers compared to the R4 reinforcement layer (almost non-existent) in the incremental shear strain plots for ROI (i) and (ii) at the 0.4g excitation level (Figure 5-16 & Figure 5-18). This exhibits a shift in the location of strain development from the higher reinforcement layers at the initial excitation levels to the lower reinforcement layers at the excitation levels close to failure. Similar to previous observations, these preliminary shear bands of high shear strain (20% or greater) corresponded to alternating zones of contractive and dilative strains (+/- 10% or greater) along the shear band. Note that a tighter lattice of radiating strain bands and an increase of strain band thickness developed with increasing excitation levels for all reinforcement layers. This is also observed in Bowman et al. (2011) and is an indication of the merging of strain development with increasing excitation to form an eventual failure surface.

In the 0.3g and 0.4g excitation level, for ROI (i), clear development of inclined radiating strain bands of 12-20% shear strain corresponding to mostly dilative strains of -10% or greater are observed beneath the R5 reinforcement layer, extending downwards towards the R4 reinforcement layer in the reinforced portion of the backfill. Development of these shear bands were first observed for the 0.1g excitation level at a shear strain level of 3-4%. These initial shear bands appear first to develop upwards from the R4 reinforcement end but curve to extend horizontally further into the reinforced backfill. With further excitation, the curved shear bands that originate from the R4 reinforcement end intersected with the R5 reinforcement layer and extend into the reinforced backfill at an inclined angle. Further strain development is concentrated below the R5 reinforcement layer from the 0.3g excitation level onwards with no vertical strain band observed to connecting these strain bands with the R4 reinforcement layer. Development of these inclined strain bands in the reinforced backfill portion is typically observed at later stages of excitation compared to those developed in the unreinforced backfill area. This may be due to the difference in stiffness within the reinforced soil block compared to the unreinforced backfill portion due to the presence of the geogrid layers. The
development of an inclined preliminary shear band extending into the reinforced backfill from the R4 reinforcement end was also observed in Bowman et al. (2011) for a 90% backfill density model.

At the 0.4g excitation level, between the lower two reinforcement layers (R1 & R2), greater strain development at the R2 reinforcement layer is observed with three separate preliminary shear bands of 20% shear strain or greater developed compared to a single preliminary shear band of an equal length and strain level observed at the R1 reinforcement layer. This corresponded to dilative strains of -10% (or greater) observed along the inclined preliminary strain bands at the R2 reinforcement layer with patches of contractive strains developed along the dilative zones. The development of the three separate preliminary shear bands is indicative of ‘competition’ for dominant strain development between the preliminary shear bands as also observed in Rechenmacher (2006) and Bowman et al. (2011).

At the wall toe, strain development beneath the R1 reinforcement layer is only observed after the 0.4g excitation level. Shear strain of about 8 - 10% and contractive strains of 6 – 8% were observed in ROI (iii) with the strain development mainly concentrated directly beneath the R1 reinforcement layer at a distance of 130mm from the GRS wall toe. Note that in the 0.4g excitation level, no clear trend of strain development was observed.

For the 0.5g excitation level, development of shear bands with high shear strains (20% shear strain or greater) is observed in both the backfill interface and wall toe regions of the backfill (ROI (ii) & (iii)). However, at this stage, it is clear that strain development is concentrated along the dominant failure plane at the R1 reinforcement layer. The dominant failure plane was formed by the merging of previously observed radiating strain bands at the R1 reinforcement end in the 0.4g excitation level. This phenomenon is not observed for the R2 reinforcement layer as a lattice-like structure of separate shear bands is still present. This difference in strain development between R1 and R2 reinforcement layers observed in ROI (ii) is indicative of a shift in strain development from the R2 reinforcement layer to the R1 reinforcement layer within the 0.5g excitation level. The occurrence of this shift is further investigated in cyclic deformation strain plots in Section 5.4.2.4.

The formation of the dominant failure plane corresponded to the development of a band of alternating zones of contraction and dilation, reaching contractive and dilative strain levels of +/- 10% or greater. These alternating zones of contraction and dilation along the failure plane indicates that critical state is reached along the failure plane (ie. shearing is occurring along the failure plane at a constant volume). For the radiating shear bands observed at the R2 reinforcement layer, multiple dilative strain bands (of -10% strains or greater) are observed to develop along these shear bands. In line with observations made in ROI (i) during the 0.4g excitation level, contractive strains (10% or greater) appear to develop as a diffuse distribution of patches in ROI (ii).

Also in ROI (ii) after the 0.5g excitation level, fully developed multiple vertical shear bands of 20% shear strain or greater, extending between the R1, R2 and R3 reinforcement ends were observed. This vertical strain development is similar to the development of radiating shear bands at the R2 reinforcement layer (mentioned in the previous paragraph) with no clear, single dominant failure plane developed. Volumetric strain plots show the development of dilative strain bands along these shear bands with contractive strains developed as a diffuse process similar to that observed for the R2 radiating strain bands. No indication of development of vertical strain bands were observed in the previous excitation levels. It is likely that the preliminary shear band bifurcated into the two
separate branches of shear strain accumulation within the 0.5g excitation level which was also observed by Bowman et al. (2011).

At the wall toe, the 0.5g excitation level brought about the development of an inclined dominant failure plane of 20% shear strain or greater (with the corresponding band of alternating zones of contraction and dilation) commencing 50mm from the wall toe. As previously stated, these alternating bands of contraction and dilation along the failure plane indicate that shearing is occurring at critical state, under constant volume. The inclined failure plane extended upwards at an inclined plane and exited the right-side of ROI (iii) at the height of approximately 120mm from the base of the wall model. Connection of this inclined failure plane at the wall toe and the inclined failure plane in ROI (ii) is likely to have occurred through the extension of the failure planes along the R1 reinforcement layer at this excitation level (critical acceleration level). This would have led to the formation a resultant failure plane extending form the backfill surface to the wall toe, forming the two-wedge failure mechanism proposed by Horii et al. (1994). This failure plane is illustrated in a sketch previously shown in Figure 5-1.

Furthermore, several smaller shear strain bands of 20% shear strain or greater were also developed in the vicinity of the failure plane. These strain bands radiate from the failure plane and appear to be random in nature with no determined point of origin or angle of inclination. The strain bands are likely part of initial radiating shear bands that merged to form a failure plane, similar to the strain bands that developed below the failure plane at the R1 reinforcement layer in Figure 5-18. Volumetric strain plots indicate a concentration of dilative strains in the bottom portion of ROI (iii) with little or no dilative strains observed in the top portion and vice versa for contractive strains developed.

### 5.4.2 In-depth analysis of strain development based on cyclic deformation strain plots

From Section 5.4.1, five key features of strain development are identified and are further discussed in the following sub-sections.

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<td>Horizontal accumulation of strain along reinforcement length</td>
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<td>4 ROI (ii)</td>
<td>R3, R2 &amp; R1</td>
<td>0.5</td>
<td>Development of failure plane at the R1 reinforcement layer</td>
</tr>
<tr>
<td>5 ROI (iii)</td>
<td>R1</td>
<td>0.5</td>
<td>Development of failure plane at the wall toe at GRS wall failure</td>
</tr>
</tbody>
</table>

To provide a greater in-depth understanding of key features of strain development, cyclic deformation plots within a single excitation level of interest are presented. As strain ranges determined from GeoPIV analysis of cyclic deformation are much smaller than strain ranges determined from residual deformation analysis, finer detailing of strain development can be shown through the cyclic deformation plots. Cyclic displacement plots can be used as a tool to determine
the commencement of key features of strain development in terms of number of cycles within a particular excitation level and they also enable further new features of strain development to be uncovered through more detailed strain plots.

To produce cyclic deformation plots, each image for GeoPIV analysis was chosen to correspond to a cyclic peak in the GRS wall panel displacement within the excitation level; a total of 50 displacement cycle peaks were recorded for each excitation level. In other words, the cyclic deformation plots show backfill deformation caused by the peak displacement of the GRS wall within a single excitation level. Initially, the cyclic strain development at every 10th cycle is presented from the 1st cycle peak to the 50th cycle peak. In some cases, smaller incremental steps are needed to show the strain development and in these cases, cyclic strain development at every 2nd cycle peak is shown.

Incremental strain development at each cycle peak was determined to be very dispersed with small strains observed in the incremental cyclic strain plots. This indicated that strain development within an excitation level is due to an accumulation of strain from each displacement cycle peak rather than a sudden increase in strain development at a single displacement cycle peak. As a result, the incremental cyclic strain plots are not presented in this section as the cumulative cyclic strain plots are deemed sufficient for analysis. The cumulative cycle strain plots presented in Figure 5-22 to Figure 5-37 exhibit the accumulation strain within a given stage of excitation. Note that a near constant increase in the number of strain bands is observed with increasing number of excitation cycles indicating relatively constant strain development within the backfill within an excitation level.

In this section, the in-depth analysis of individual ROIs is performed. For this reason, the shear and volumetric strain ranges in the following strain plots are varied to suit each individual key strain development feature in question. Note that when determining the strain range (i.e. maximum strain levels) in these strain plots, a balance between displaying the maximum strain level achieved along the shear band and avoiding saturation of the strains developed in the backfill has to be made.

**5.4.2.1 Formation of preliminary strain band at the R4 and R5 reinforcement ends at 0.1g (ROI (i))**

In the following figures, the optimum shear strain and volumetric strain range are chosen to be 0 to 20% and +/- 10% strains respectively. Different strain ranges were trialled which resulted in the determination of these optimum strain ranges. For example, along the preliminary shear band, we found through initial analysis that the strain developed did not reach 30% shear strain (about 20% shear strain); therefore a shear strain range up to 20% was chosen for better clarity of the strain development in the surrounding backfill. Similar premise was taken in determining the optimum strain range for volumetric strain plots.

Figure 5-22 and Figure 5-23 show that the inclined preliminary shear band already shows some level of strain development of up to 6% shear strain in the 10th cycle peak. The 6% shear strain development corresponded to some contractive strain development of up to 2-3% in the volumetric plots. Gradual strain development over the cycle peaks is observed with the preliminary strain band of 20% shear strain or greater only fully developed at the end of the excitation level, in the 50th cycle peak. Contractive strains reaching 10% volumetric strain or greater developed in small patches along the preliminary strain band in the 40th cycle peak. Some dilative strains have formed at this stage of excitation but not as significantly as contractive strains and form in a band instead of in patches as previously observed in residual strain plots.
Figure 5-22: Cumulative shear strain plots (in %) for KL7 within ROI (i) at the 0.1g excitation level - 1,10,20,30,40,50th displacement cycle peaks.

Figure 5-23: Cumulative volumetric strain plots (in %) for KL7 within ROI (i) in the 0.1g excitation level - 1,10,20,30,40,50th displacement cycle peaks.

Note that disappearance of strain bands for the cumulative strain plots, indicating reduction of shear strains, is observed within ROI (i) (above the R4 reinforcement layer between the 30th and 40th cycle peak). This is likely due to the cyclic nature of loading resulting in occurrences of cyclic shearing within the backfill. This could be related to threshold cyclic shear strain amplitude, where if the cyclic strain amplitude is below this threshold strain, the soil behaves as a perfect linearly elastic material with irreversible strains only developing beyond this threshold strain (Vucetic, 1994). However, Vucetic (1994) found this threshold strain level to be very low of a strain magnitude of less than 1%. Reduction of shear strains in the initial stages of an excitation level at displacement cycle peaks were also observed in Jackson (2010) which stated that it is possible that the soil response in the first few cycles of excitation is not initially well-defined.
5.4.2.2  **Horizontal accumulation of strain along reinforcement length in the backfill interface in 0.1g (ROI (i) & ROI (ii))**

In this section, the horizontal accumulation of strain along the length of the reinforcement layers is focused on. Due to the high strain development along the reinforcement lengths early in the 0.1g excitation level, the volumetric strain range is increased from its typical +/- 10% range to +/- 20% with the shear strain range maintained at 0-20% strain.

From the following figures, a progressive strain development along the reinforcement lengths is observed between the three reinforcement layers with the top R3 reinforcement layer in the ROI experiencing the earliest strain development along its length. Figure 5-24 show that by the 4th displacement cycle peak, shear strains of 20% or greater corresponding to contractive strains of 20% or greater were achieved along the length of the reinforcement layer R3. At this stage, lower magnitude strains of 8% shear strain and 10% contractive strains is observed for the lower R2 reinforcement layer while the R1 reinforcement layer developed minimal strain development in the 4th displacement cycle peak. This observation indicates a tiered development of strain between reinforcement layers with the higher reinforcement layers experiencing a greater strain development at a given time. This is expected as the primary displacement mode of the FHR wall is overturning which will affect the higher reinforcement layers more significantly.

In Figure 5-25, contractive strains appear to develop with a similar magnitude as shear strain along the reinforcement length (20% or greater achieved in the R3 reinforcement layer by the 4th cycle peak). This is different to typical observations where alternating zones of contractive and dilative strains are observed along a shearing plane due to the attainment of critical state along the shear plane. The contractive nature of the shear band formed along the reinforcement length is likely due to the physical gaps between the geogrid reinforcement and the surrounding backfill that reduce due to the pullout engagement of the reinforcement.

![Figure 5-24: Cumulative shear strain plots (in %) for KL7 within ROI (ii) in the 0.1g excitation level - 2,4,6,8,10,12th displacement cycle peaks.](image-url)
Similar observations were made in Giang et al. (2010) who performed cyclic pullout tests on geogrids and found contractive behaviour occurred behind the transverse ribs and dilative behaviour occurred in front of the transverse ribs. The lack of dilative behaviour in the wall models in comparison could be due to the relatively low thickness of the ribs (less than 0.3mm) compared to the 1mm thickness of the geogrid used by Giang et al. (2010). A lower thickness resulted in little bearing resistance of the geogrid and hence relatively minimal dilative strain in comparison to contractive strains which developed behind the transverse ribs.

5.4.2.3 Strain development between the three preliminary strain bands in 0.3g and 0.4g (ROI (ii))

To distinctly show the development of the three preliminary strain bands, the shear strain plots for the 0.3g and 0.4g excitation level are shown with a maximum shear strain level of 10% in Figure 5-26 and Figure 5-28. This is because during the preliminary analysis, we found that the preliminary shear bands displayed shear strain development of up to 8-10% strains within the 0.3g and 0.4g excitation level. Similarly, due to the low volumetric strains developed along the preliminary strain bands in these two stages, the volumetric strain range is limited to +/- 5% in Figure 5-27 and Figure 5-29. Although this results in a thicker contractive strain band shown along the reinforcement length, this is not touched on as it has been previously discussed in Section 5.4.2.2.

To understand this bifurcation of the three preliminary strain bands at R2, the cyclic strain development within the 0.3g excitation level is first presented in Figure 5-26. Figure 5-26 shows that at the R2 reinforcement layer (at this point, the location of maximum strain development), the formation of the top preliminary shear band (S1) first occurred at the 10th and 20th cycle peak with a distinct shear band (8% shear strain) observed in the 40th cycle peak. In the following 50th cycle peak, clear development of the middle shear band (S2) up to a similar shear strain of 8% is then observed. Although the initial formation of the S3 shear band is first observed in the 30th cycle peak, at this stage, the S3 shear band is still in its infancy with 4% shear strains accumulated at the end of the excitation level.
From Figure 5-27, we observe that the corresponding volumetric strain developed at the location of the preliminary shear bands is relatively small with -4% (dilative) to 4% volumetric strain bands developed discretely in parallel to one another in the 50th cycle peak. Note that the preliminary shear bands corresponded to the contractive strains developed with dilative strains developed alongside the preliminary shear bands. This agrees well with our expectations that the backfill will be initially contractive during the initial stages of shearing due to its loose nature. The initial formation of the S1 shear band and subsequent development of the deeper shear bands with increasing displacement cycle peaks indicates a gradual progression of shear strain development to the deeper shear bands. This is even more pronounced in the 0.4g excitation level (shown in Figure 5-28 and Figure 5-29) where significant development of the shear band at the R1 reinforcement layer, S4 is observed.

Figure 5-26: Cumulative shear strain plots (in %) for KL7 within ROI (ii) in the 0.3g excitation level - 1,10,20,30,40,50th displacement cycle peaks.

Figure 5-27: Cumulative volumetric strain plots (in %) for KL7 within ROI (ii) in the 0.3g excitation level - 1,10,20,30,40,50th displacement cycle peaks.
At the following 0.4g excitation level, initially, strain development of the S2 shear band is the most pronounced among the three preliminary shear bands with early development observed at the 10th cycle peak and a distinct strain band of 8 to 10% shear strain forming at the 30th cycle peak (Figure 5-28). Similarly, the similar development of the distinct S1 and S3 shear band is also observed but to a later stage (at the 40th cycle peak for S3 and 50th cycle peak for S1). These observations indicate a shift of position of dominant shearing from the higher S1 shear band at the 0.3g excitation level to the deeper S2 shear band at the 0.4g excitation level. Similar to observations at the 0.3g excitation level, the volumetric strains developed as distinct shear bands of -4% (dilative) and 4% volumetric strain that form parallel to one another (Figure 5-29). This is observed earlier in the 0.4g excitation level (by the 20th cycle peak) in comparison to the 0.3g excitation level (by the 50th cycle peak).
At the 0.4g excitation level, significant formation of another shear band (S4) at the R1 reinforcement end is observed with strains up to a similar shear strain level as S2. These observations indicate a progressive shift of strain development to deeper shear surfaces (bands) with continual model excitation which was also observed in Jackson (2011). The development of the S4 shear band is further focused on in the next Section 5.4.2.4. Note that, no indication of the merging of strains to form a distinct dominant strain band with alternating zones of contraction and dilation (which is indicative of the attainment of critical state and development of the dominant failure plane as previously observed in the residual deformation plots) is observed at the R1 and R2 reinforcement ends in the two stages of excitation.

**5.4.2.4  Development of failure plane at the R1 reinforcement layer in 0.5g (ROI (ii))**

At the 0.5g excitation level, the development of a failure plane (S4) extending from the end of the R1 reinforcement layer up to the surface of the backfill is observed. From cyclic strain plots of within the 0.5g excitation level, we observe full development of the S4 strain band extending beyond ROI (ii) by the 20th displacement cycle peak, forming the failure plane within the backfill interface (The failure plane is characterised by the concentration of shear strains along a well-defined plane in the unreinforced backfill (30% shear strain or greater was achieved) and the development of alternating zones of contractive and dilative strains in corresponding volumetric strain plots (+/- 15% volumetric strain or greater) as observed in Figure 5-30 and Figure 5-31.

Although the S4 failure surface was well developed by the 10th displacement cycle peak, further development of radiating strain bands originating from the failure plane is observed in the following displacement cycle peaks. These radiating strain bands form a lattice-like structure also observed in residual strain displacement plots. This observation indicates that despite a high concentration of strain development along the failure plane, strain development is not solely concentrated along the failure plane.

**Figure 5-30: Cumulative shear strain plots (in %) for KL7 within ROI (ii) in the 0.5g excitation level - 1,10,20,30,40,50th displacement cycle peaks.**
Figure 5-31: Cumulative volumetric strain plots (in %) for KL7 within ROI (ii) in the 0.5g excitation level - 1,10,20,30,40,50th displacement cycle peaks.

Note at the R2 reinforcement layer, shear strain development is mainly concentrated along the plane of the initial S1 shear band (previously mentioned in Section 5.4.2.3) in the 0.5g excitation level. This shear band at the R2 reinforcement layer formed an identical angle of inclination as the dominant failure plane at R1. It is likely that development was eventually concentrated along the S1 strain band because it was formed at a similar angle of inclination as the failure plane (the S2 and S3 shear bands were at shallower angles). To better understand the development of strain in the initial stages of the 0.5g excitation level, cyclic deformation plots for the first 5 cycles of the 0.5g excitation level are shown in Figure 5-32 and Figure 5-33. Due to the relatively low strain developments in the first five cycles, the strain ranges for the following strain plots are limited to 0-10% and +/- 5% for shear and volumetric strain for greater clarity.

Figure 5-32: Cumulative volumetric strain plots (in %) for KL7 within ROI (ii) in the 0.5g excitation level – 0,1,2,3,4,5th displacement cycle peaks.
In the first two displacement cycles, relatively minimal strain development is observed at the ends of the three reinforcement layers. In the 3rd displacement cycle peak, only strain development of the S1 and S4 strain bands (up to 4% shear strain) is observed without other strain bands developed at this stage. In the 4th and 5th displacement cycle peaks, strain development is mainly concentrated along the S4 strain band as indicated by the initial formation of the failure plane at R1 of 10% shear strains or greater (found to be approximately 12% shear strains at this stage as observed in strain plots limited to a maximum shear strain of 20%). This implies dominant strain development at the R1 reinforcement layer very early on in the 0.5g excitation level (a shift from that observed in the 0.4g excitation level where strain development was still concentrated at the R2 reinforcement layer). Note that in the first 5 cycle peaks, the failure plane corresponds to development of dilative strains in between contractive strain bands with small patches of contractive strains becoming visible along the dilative band in the 5th displacement cycle peak.

### 5.4.2.5 Development of failure plane at the wall toe in 0.5g (ROI (iii))

In residual displacement plots, the development of an inclined failure plane at the wall toe is observed to develop within the 0.5g excitation level. To better understand this development, the strain development within ROI (iii) during the 0.4g and 0.5g excitation levels is presented in Figure 5-34 to Figure 5-37.

An analysis within the 0.4g excitation level was performed to determine if any occurrence of small strain development along the eventual failure plane could be detected at this level. Figure 5-34 and Figure 5-35 show that only small strain development (up to shear strain levels of 8% and dilative and contractive strains of +/- 5% or greater) occurred within the 0.4g excitation level even in the 50th displacement cycle peak. Note that relatively higher shear strain development corresponding to mainly contractive strains is observed along the R1 reinforcement layer (top right area of ROI (iii)) which has been discussed in Section 5.4.1.1 and 5.4.2.2. At this stage of excitation, no clear shear band is observed with strain development occurring throughout the entire ROI (iii).
For the 0.5g excitation level, strain development in the initial displacement cycle peaks are presented in Figure 5-36 and Figure 5-37 as the full development of the failure plane in ROI (iii) within the 12th displacement cycle peak was observed. High strain levels were achieved early on in the excitation level with strain development mainly concentrated along the eventual failure plane (~8% shear strain was observed by the 6th displacement cycle peak). By the 10th displacement cycle peak, a clear and distinct failure plane of 20% shear strain or greater which corresponded to a volumetric strain band of alternating dilative and contractive strains of +/- 10% or greater is observed.

Note that although strain development was concentrated along the eventual failure plane (persistent shear band), strain development is also observed in the surrounding backfill before and after full development of the failure plane. This is similar to observations made for ROI (ii) where strain development continued in the form of radiating strain bands from the full-developed failure plane. Furthermore, minimal amount of strain development is observed along the length of the R1 reinforcement layer compared to the amount of strain development along the failure surface. This
indicates that at the 0.5g excitation level, the majority of strain developed at the wall toe is concentrated along the eventual failure plane which would have connected with the failure plane formed along the R1 reinforcement layer and the dominant inclined failure plane in ROI (ii).

Similar reduction of shear strains was previously observed and discussed in Section 5.4.2.1 for ROI (i). This event is likely due to the cyclic nature of loading and is typically observed in the initial cycles of excitation where it is likely that the soil response is not well-defined (Jackson, 2010).

Figure 5-36: Cumulative shear strain plots (in %) for KL7 within ROI (iii) in the 0.5g excitation level - 2,4,6,8,10,12th displacement cycle peaks.

Figure 5-37: Cumulative volumetric strain plots (in %) for KL7 within ROI (iii) in the 0.5g excitation level - 2,4,6,8,10,12th displacement cycle peaks.
5.5 **Comparison between different GRS wall models tested**

In this section, comparisons of shear strain development at specific excitation levels are made between the different GRS models tested and the representative wall model, KL7. Key features of strain development such as the location of strain bands and the level of cumulative strain development at the end of each excitation level are discussed for each of the model comparisons made. Key comparisons of strain development of different wall models are made at specific excitation levels chosen depending on the models in focus. The specific excitation levels are chosen to best illustrate two stages of model excitation.

In Stage (i), typically the 0.1g to 0.2g excitation level, seismic excitation applied has minimal impact on the overall model with only slight wall displacement typically experienced by the GRS wall model (<10mm for the wall models (~1% wall height)). Full interlock between the geogrid reinforcement and soil particles is developed within this stage of excitation as some small displacement is needed for full interaction to be achieved. In Stage (ii), typically the 0.4g or 0.5g excitation levels, the stability of the wall has been compromised due to the continual excitation applied. Wall displacement at this stage of excitation is significantly greater (40-80mm for wall models, approximately 4-9% wall height). Within the backfill, a progression of deeper shear surfaces with increasing base excitation is observed with the initial formation of the failure plane observed at the R1 reinforcement end.

5.5.1 **Influence of staggered reinforcement layout**

To determine the effect of a staggered reinforcement layout on the backfill deformation, wall model KL5 which had a uniform reinforcement length ratio of 0.75 is compared with wall model KL8 which had longer reinforcement for its top two reinforcement layers with L/H=0.9. A maximum shear strain level of 20% and corresponding volumetric strain range of +/- 10% strains were selected to enable comparisons to be made between excitation levels without significantly losing strain detail.

From Figure 5-38 and Figure 5-39, greater strain development is observed for the KL5 model than for KL8 at 0.2g excitation. This is indicated by the start of an inclined preliminary shear band at the R4 reinforcement layer, a higher number of radiating shear bands developed parallel to the preliminary shear band at the R4 reinforcement layer and a clearly developed shear band of 20% shear strain or greater and 10% contractive strain or greater along the R1 reinforcement layer at the wall toe. These observations indicate that the staggered reinforcement KL8 wall model experiences reduced internal deformation as a result of the increased reinforcement lengths. This is in agreement with wall displacement results presented and discussed in Chapter 4. However, it is noted that the strain development between the two wall models in ROI (ii) is relatively similar. This is likely due to the similar reinforcement lengths in ROI (ii) (L/H = 0.75) between the two wall models.

An important difference in strain development between the two wall models is the lack of radiating shear bands for the KL8 wall model in ROI (i). These typically form parallel to the preliminary shear band at the R4 reinforcement layer in the unreinforced backfill portion. These shear bands originate from the lower R3 reinforcement layer and is part of a preliminary shear band that strives to extend to the backfill surface. For KL5, these shear bands intersect the plane of the R4 reinforcement layer at the x-coordinate of 800 (800mm horizontally from wall toe). However, the R4 reinforcement extends up to the x-coordinate of 810 in KL8, preventing these shear bands from extending upwards to form an inclined shear band up to the backfill surface. This is further illustrated in Figure 5-40.
Figure 5-38: Cumulative shear strain plots (in %) within all ROIs at the completion of the 0.2g excitation level:
(A) KL5 (LH075); (B) KL8 (StagLH).

Figure 5-39: Cumulative volumetric strain plots (in %) within all ROIs at the completion of the 0.2g excitation level:
(A) KL5 (LH075); (B) KL8 (StagLH).

Figure 5-40: Cumulative shear strain plots (in %) in ROI (i) at the completion of the 0.3g excitation level:
(A) KL5 (LH075); (B) KL8 (StagLH).

Figure 5-40 shows that instead of forming parallel to the R4 inclined shear band, the R3 inclined shear band connects with the R4 inclined shear band and forms a single inclined band that connects both R4 and R3 reinforcement ends. This development is likely due to the geometry of the reinforcement layout. If the R4 reinforcement layer was longer (e.g. 900mm), it is likely that a single
shear band between the two reinforcement ends will not form. Instead, after interception by the R4 reinforcement layer, the R3 shear band will develop along the R4 reinforcement length before merging with the R4 inclined shear band and extending towards the backfill surface. This effect of longer top reinforcement lengths would have contributed positively to wall stability due to the additional resistance to shear band development within the backfill in the initial stages of excitation.

In Figure 5-41 and Figure 5-42, it is observed that the distinct failure plane at the R1 reinforcement end in the non-staggered KL5 model has fully developed reaching 20% shear strain or greater and a corresponding thickness of approximately 10mm which extends beyond ROI (ii). This failure plane is also characterised by the continuous strain band of alternating zones of contraction and dilation in volumetric strain plots. Comparatively for KL8, reduced strain development along the KL8 wall model’s eventual failure plane is indicated by a non-continuous shear band in ROI (ii) (Figure 5-41). Similar observations can be made about strain development along the inclined failure surface located within ROI (iii). This once again illustrates the higher wall stability achieved in the staggered reinforcement wall model. For the 0.4g excitation level, displacements within ROI (i) were too large for the GeoPIV analysis to accurately identify the strains, leading to nonsensical plots as shown.
5.5.2 Influence of backfill density

To determine the effect of soil density on the pre-failure deformation of the backfill, wall model KL4 with a relative density of 85% is compared to KL5 which was constructed with an identical reinforcement layout but at a lower relative density of 50%. Strain development in ROI (ii) and (iii) only is compared between the two models due to a combination of reflective sunlight within ROI (i) (as stated in Section 5.3.4.5) and incorrect exposure (inadequate lighting) when capturing the high-speed images during testing for the KL4 wall model. The incorrect exposure settings also affected the images captured of ROI (ii) and are the reason for the apparent strain development along the reinforcement lengths. Despite this, clear differences between the two models are still observed.

From Figure 5-43 and Figure 5-44, it is observed that the higher density KL4 model exhibits a significantly lower strain development within the unreinforced backfill compared to the KL5 wall model with little to no radiating shear bands to form a lattice observed in the KL4 model. Furthermore, in ROI (iii), the higher density KL4 model does not develop distinct strains along the R1 reinforcement length. Comparatively, the lower density KL5 model experiences distinct strain development along the R1 reinforcement layer of approximately 8% shear strain and 10% contractive strain or greater. This is an indication of the lower sliding displacement experienced by the KL4 wall model as previously observed in Chapter 4. However, note that along each reinforcement length in ROI (ii), like the KL5 model, the higher density KL4 model also exhibits high strain development, achieving shear strains and contractive strains of 20% (from previous strain plots analyses) and 10% or greater. Similar strain development between the wall models is observed along the reinforcement length is likely because strain development along the reinforcement lengths is less a function of backfill density but is dependent on the physical gaps between the reinforcement and backfill induced by the reinforcement displacement.

![Figure 5-43: Cumulative shear strain plots (in %) within ROI(ii) and (iii) at the completion of the 0.1g excitation level: (A) KL4 (DR85); (B) KL5 (DR50).](image)
The reduced strain development in ROI (ii) for the KL4 wall model indicates the greater stability of the wall model which is due to the significantly greater geogrid pullout resistance as a result of the higher density backfill. Although similar strains were developed along the reinforcement length, the pullout resistance and subsequent stabilising force generated in the KL4 wall model was significantly greater, resulting in minimal wall displacement experienced which is indicated by the insignificant strains developed within the backfill. Comparatively, the pullout resistance of the lower density KL5 model is significantly lower thus resulting in preliminary shear bands developed in the backfill within the 0.1g excitation level (due to greater backfill deformation).

To determine the differences between the two models just before failure (Stage (ii)), strain development of the wall models just before each wall model’s failure excitation level is shown in Figure 5-45 and Figure 5-46. Strain ranges are increased to 0-30% for shear strain and +/-15% for volumetric strain in the figures due to large strains developed in the backfill at this excitation level.

For the KL4 wall model with higher backfill density, strain development is observed mostly along the plane of the dominant failure surface with relatively little strain developed in the surrounding backfill. This is significantly different to lower density wall models that typically develop a wide lattice of strain. Due to the higher backfill density in KL4, strains are unable to develop freely and as a result, strains are concentrated along a reduced number of potential failure planes in the backfill. Note that the dominant failure plane in each wall model appears to develop at a similar angle of inclination with the lower density model developing only a slightly shallower failure plane.

For volumetric strains between the two wall models, as shown in Figure 5-46, the lower density KL5 model displays a significantly greater proportion of contractive strains to dilative strains compared to the higher density KL4 model. This observation is more pronounced within the reinforced portion of the backfill in ROI (ii) where large zones of dilation are observed in the KL4 wall model. This is expected as a relatively dense soil will typically dilate after some small initial compression at the start of shearing whereas looser soils will typically experience contraction. Note that, alternating zones of contraction and dilation are observed along the failure surface indicating that for both wall models, the soil has been sheared to the critical state condition.
5.5.3 Influence of 3kPa surcharge

To determine the effect of surcharging on the strain development within the backfill, wall model KL10 which had a 3kPa backfill surcharge is compared with the representative wall model KL7. Both wall models were constructed with the same reinforcement length ratio, L/H of 0.9 and at the same backfill relative density of 50% with failure of both models achieved at the 0.6 excitation level.

Firstly, strain development between the two models during the initial stages of excitation (Stage (i)) is compared. Figure 5-47 shows that at the completion of the 0.2g excitation level, the surcharged KL10 wall model exhibited less strain development in comparison to the non-surcharged counterpart in all three ROIs. This is indicative of reduced wall deformation in the initial stages of excitation due to the application of the surcharge providing a stabilising effect as previously discussed in Section

Figure 5-45: Cumulative shear strain plots (in %) within ROI(ii) and (iii) at the completion of the 0.5g excitation level: (A) KL4 (DR85); (B) KL5 (DR50).

Figure 5-46: Cumulative volumetric strain plots (in %) within ROI(ii) and (iii) at the completion of the 0.5g excitation level: (A) KL4 (DR85); (B) KL5 (DR50).
4.4.1.3. The surcharge increased the normal stresses applied to each reinforcement layer thus increasing the soil-geogrid interaction leading to higher stability of the GRS wall model.

Another observation is a slight reduction in observed dilative strains developed within the KL10 surcharged model (Figure 5-48). This observation is more apparent in ROI (i) which is closer to the backfill surface and experiences the greatest increase of confining stress due to the surcharge. An increase in a soil’s effective stress will reduce its critical state void ratio; resulting in its diminishing potential for dilation (i.e. the soil becomes more contractive). This demonstrates the ability of the GeoPIV analysis to accurately determine the volume change of the backfill during shearing.

![Figure 5-47 Cumulative shear strain plots (in %) within all ROIs at the completion of the 0.2g excitation level: (A) KL7 (no surcharge); (B) KL10 (3kPa surcharge).](image)

![Figure 5-48: Cumulative volumetric strain plots (in %) within all ROIs at the completion of the 0.2g excitation level: (A) KL7 (no surcharge); (B) KL10 (3kPa surcharge).](image)
Figure 5-49 and Figure 5-50 show the shear and volumetric strain development of both the KL7 and KL10 wall models in each of the ROIs in the later stages of excitation at the 0.4g excitation level. Firstly, it is observed that for all ROIs, the surcharged KL10 model exhibits greater strain development in the form of a greater number of shear bands. Furthermore, development of the vertical shear bands only observed in the KL7 model at the 0.5g excitation level is already observed in the 0.4g excitation level for the KL10 model. This indicates that the KL10 model has undergone a greater progression to failure compared to its non-surcharged counterpart. At this stage of excitation, a combination of increased active earth pressure, increased shear stresses within the backfill due to dynamic loading and contribution of the surcharge to the vertical stresses to the failure block acts to destabilise the GRS wall. The greater deformation of the surcharged KL10 model observed in the later stages of excitation is because of the dominance of the destabilising effect of the surcharge over the stabilising effect of the surcharge (increasing the pullout resistance of the geogrid through increased soil-geogrid interaction).

Figure 5-49: Cumulative shear strain plots (in %) within all ROIs at the completion of the 0.4g excitation level: (A) KL7 (no surcharge); (B) KL10 (3kPa surcharge).

Figure 5-50: Cumulative volumetric strain plots (in %) within all ROIs at the completion of the 0.4g excitation level: (A) KL7 (no surcharge); (B) KL10 (3kPa surcharge).
However, in the non-surcharged KL7 model, development of the inclined preliminary shear band of 20% shear strain or greater in ROI (i) observed at the R4 reinforcement end is not observed in the surcharged KL10 model despite greater strain development in the region (Figure 5-49). On the other hand, at the R2 reinforcement end at the 0.4g excitation level, strain development along a distinct preliminary shear band is already observed for the surcharged KL10 model while competition of strain development among three separate preliminary shear bands is observed in the non-surcharged KL7 model.

Typically, progression of deformation involves the downward propagation of inclined shear bands that extend from the backfill surface to the ends of the reinforcement until the final and lowest shear band (eventual failure surface) is developed at the ends of the lowest reinforcement as previously observed by Jackson (2010). The lack of a preliminary shear band in the surcharged model at the top two reinforcement layers indicate that a more rapid progression of deformation towards the eventual failure surface at the R1 reinforcement end occurred for the surcharged model. The surcharge appears to cause the position of development of the initial preliminary shear band (typically formed at the top two R4 and R5 reinforcement layers) to shift downwards to the end of the R3 reinforcement layer. This could be due to the increased pullout resistance imparted to the first two reinforcement layers R5 and R4 due to the application of surcharge to the backfill.

5.6 Summary

The development of strains within the backfill of the GRS wall models under seismic excitation were shown in this chapter through strain plots resulting from the GeoPIV analysis. High speed images of the three regions of interest within the backfill were captured for this purpose. Both shear strain and volumetric strain plots were presented from the GeoPIV analysis. Residual deformation of the backfill (at the end of each excitation level) was mainly focused on in this chapter with cyclic deformation plots (deformation within an excitation level) used to supplement key observations from residual deformation analyses. Lastly, strain development between the different wall models tested is briefly discussed.

A summary of key features of strain development for the representative GRS wall model (failure excitation level of 0.6g) is presented:

- Accumulation of shear and contractive strains of 20% and 10% or greater along the reinforcement length for all reinforcement layers in the backfill interface windows within the 0.1g excitation level with reduced strain development in subsequent excitation levels. This strain development is likely caused by the pull-out displacement of the reinforcement layers under excitation and subsequent contraction due to physical gaps between the geogrid and surrounding backfill.

- Further in-depth analysis shows a progressive strain development along the reinforcement lengths with increasing depth which is expected as the overturning displacement of the FHR wall panel would affect the higher reinforcement layers more significantly.

- Development of a preliminary shear band of 20% (or greater) shear strain was observed at the 0.1g excitation level for the top two reinforcement layers. This indicates greater strain development for the top reinforcement layers. Alternating zones of contractive and dilative strains of +/- 10% (or greater) are observed to have developed along the length of the
preliminary shear band with contraction developing as a relatively more diffuse process compared to dilation.

- In the 0.2g excitation level, radiating strain bands originating from the preliminary shear band were developed in a lattice-like structure. In the initial stages of excitation, only contractive strains were developed along the radiating strain bands. With increasing excitation levels, a tighter lattice of radiating shear bands and a decrease in strain band thickness developed; an indication of consolidation of strain development. The merging of the radiating strain bands to form a preliminary shear band of high shear strain (20% strain or greater) is generally observed at all reinforcement ends but extension of this preliminary shear band to form a dominant failure plane is only observed in the R1 reinforcement layer.

- A shift in the location of strain development from the higher reinforcement layers (R5, R4 and R3) to the lower reinforcement layers (R2 and R1) was observed in the 0.3g and 0.4g excitation levels. Despite no further development of the preliminary shear band (of 20% strain or greater), an increase in number of radiating shear bands at R3 and R4 was observed in these excitation levels.

- In the 0.4g excitation level, greater strain development at the R2 reinforcement layer is observed with three separate preliminary shear bands of 20% shear strain developed compared to a single preliminary shear band of an equal length observed at the R1 reinforcement layer. However, cyclic deformation plots show that the development of the preliminary shear band at R1 was relatively similar to the development of preliminary strain bands at the R2 reinforcement layer; indicating a progressive shift to strain development to deeper shear surfaces with continual model excitation.

- In the 0.5g excitation level, strain development is observed to be focused along the dominant failure plane which has formed at the R1 reinforcement layer. Development of the dominant failure plane at R1 indicates a shift in strain development from the R2 reinforcement layer in 0.4g excitation level to the R1 reinforcement layer in the 0.5g excitation level. Cyclic deformation plots within the 0.5g excitation level show that development of strain was concentrated along this dominant failure plane by the 4th displacement cycle peak.

- Development of the failure plane corresponded to alternating zones of contraction and dilation of +/- 10% strain or greater along the failure plane. This indicates the attainment of critical state along the failure plane resulting in constant volume shear.

- In ROI (ii) after the 0.5g excitation level, fully developed vertical shear bands of 20% shear strain extending between the R1, R2 and R3 reinforcement ends were observed. No indication of development of vertical strain bands were observed in the previous excitation levels. It is likely that the preliminary shear band branched out into the two separate branches of shear strain accumulation within the 0.5g excitation level.

- At the wall toe, strain development beneath the R1 reinforcement layer is only observed after the 0.4g excitation level. Strain development was mainly concentrated directly beneath the R1 reinforcement layer with no clear trend of strain development observed. However, in the 0.5g excitation level, development of an inclined persistent failure plane of 20% shear strain or greater is observed. Cyclic deformation plots show that the inclined failure plane reached 20% shear strain or greater by the 8th displacement cycle peak.

- Connection of the inclined failure plane at the wall toe and the inclined failure plane in ROI (ii) through the extension of the failure planes along the R1 reinforcement layer is likely to
have occurred at the excitation level before failure (critical acceleration level) forming a resultant failure plane similar to the two-wedge failure mechanism proposed by Horii et al. (1994).

In addition, a brief summary of key observations when comparing GeoPIV results from different GRS wall models tested is presented:

- Greater strain development is observed for the LH075 wall model when compared to the StagLH wall model indicating that the StagLH wall model experiences reduced internal deformation. However, it is noted that strain development between the two wall models is relatively similar in the initial excitation levels.
- Strain development in ROI (i) indicates that the longer R4 reinforcement layer in the StagLH wall model prevented the radiating shear bands from the R3 reinforcement layer from extending upwards to the backfill surface. As a result, a single inclined shear band connecting both the R3 and R4 reinforcement ends was developed.
- This effect indicates an increased shear resistance within the backfill in the initial excitation levels due to longer reinforcement layers which would have contributed positively to wall stability. It was stipulated that if the R4 reinforcement layer was longer, the R3 shear band would have developed along the R4 reinforcement before merging with the R4 shear band.
- Significantly lower strain development was observed for the higher density KL4 (Dr=85%) model in with no radiating shear bands observed at the 0.1g excitation level. However, similar strains with the lower density KL5 (Dr=50%) model were developed along the reinforcement lengths. This is likely due to strain development along the reinforcement lengths being more dependent on the physical gaps between the reinforcement and backfill induced by reinforcement displacement rather than backfill density.
- In the excitation level prior to failure, relatively low strain development is observed in the surrounding backfill around the dominant failure plane in comparison to the lower density KL5 model. This is likely due to the higher backfill density in KL4 restricting the development of strains (higher shearing resistance) and resulting in the concentration of strains along a reduced number of potential failure surfaces.
- It is noted that the lower density KL5 wall model exhibits significantly greater contractive strains compared to KL4 which is in agreement with general principles of soil mechanics.
- In the initial excitation level (0.2g), the surcharged wall model (KL10) exhibited less strain development compared to its non-surcharged counterpart. This is due to the increased wall stability as a result of increased soil-geogrid interaction. In the later stages of excitation (0.4g), the surcharged model exhibits greater strain development due to the greater destabilising effect of the surcharge at this stage of excitation. These observations are similar to observations regarding the effect of surcharging on wall stability and showed the shift in wall stability contribution of the surcharge.
- A reduction of dilative strains within the surcharged model close to the backfill surface (ROI (i)) was observed as a result on the increased confining stress due to the surcharge.
- The absence of a preliminary shear band in the surcharged model for the top two reinforcement layers indicate that the surcharge likely caused the position of development of initial preliminary shear band (typically at R4 or R5) to shift downwards to the R3 reinforcement end. This could be due to the increased pullout resistance of the top two reinforcement layers due to the surcharge application.
5.7 References


Chapter 6.0 CONCLUSION

6.1 Overview of experimental study performed

Although good seismic performance of GRS structures has been observed in previous earthquakes, further research is required to better understand the seismic performance of GRS structures and improve design practices. Furthermore, design methods in New Zealand have not been well established with several different overseas standards and design guidelines currently used to design GRS structures in New Zealand (Murashev, 2003). As a result, GRS structures do not have a uniform level of seismic and static resistance; hence involve different risks of failure. Full-height rigid (FHR) facing type walls are focused on in this study due to their better seismic performance as indicated in Japan practices.

A series of twelve 1-g shake-table tests were performed on 1:5 reduced-scale GRS wall models with a full height rigid (FHR) facing. The key objective of the wall models tested was to determine the influence of GRS wall parameters such as the reinforcement length and layout, backfill density and backfill surcharging on the seismic performance of GRS walls. This was achieved through intensive instrumentation of the wall model in the form of earth pressure cells, reinforcement load cells, accelerometers, potentiometers and the use of high-speed cameras.

A similar wall model setup to Jackson (2010) was used with the wall model constructed within a rigid box of 3.0 m length, 0.8m width and 1.1m height. The rigid box had a 20mm thick transparent acrylic sidewall which enabled a side view of the model and observation of the development of failure surfaces within the retained backfill during excitation. High speed images captured of the retained backfill were analysed to examine the characteristics of backfill deformation and the failure planes associated with excessive displacement of the FHR wall panel. A stepped sinusoidal excitation of 5Hz and 10s duration with staged acceleration amplitude increase in 0.1g increments was applied to each wall model until failure (excessive displacement of the wall panel) occurred.

A majority of wall models tested were constructed with a relative backfill density of 50% with only three wall models constructed at approximately 85%. Densification of the backfill to the desired relative density was achieved through the use of a compactor plate and vibration of the model. Four wall models were tested with the application of a 3kPa surcharge to the surface of the retained backfill. Surcharging was achieved through a combination of steel plates and surcharge bags of steel punching. Reinforcement layouts used in this research include uniform reinforcement layouts with reinforcement length ratios (L/H) of 0.75 and 0.9 as well as a staggered reinforcement layout with longer reinforcement lengths (L/H = 0.9) for the top two reinforcement layers.

6.2 Key conclusions from experimental study

Analyses regarding the seismic performance of GRS walls have been discussed and presented in Chapters 4 and 5 of the thesis. From these analyses, key conclusions can be made regarding the seismic behaviour of a typical GRS wall and the influence of wall parameters such as reinforcement length and backfill density on the observed seismic behaviour in the wall models.
6.2.1 Behaviour of a typical GRS wall

For a typical GRS wall model, the following conclusions were made regarding the observed behaviour:

- Overturning was the dominant failure mode in all excitation levels with sliding displacement only significant in the last two excitation levels of the wall model (at the critical acceleration point of 0.4g for the staggered reinforcement and $L/H=0.75$ models and 0.5g for the $L/H=0.9$ models).

- A bi-linear displacement-acceleration curve was observed where beyond a certain threshold acceleration level (critical acceleration), the rate of displacement of the wall panel increases rapidly indicating failure (loss of stability) of the wall.

- The contribution of sliding to the total wall displacement was observed to decrease in the initial excitation levels as some displacements was required for the engagement of the geogrid and mobilization of its resistance.

- A gradual increase of acceleration amplification with increasing excitation levels was observed in the wall models due to the reduction of the backfill natural frequency resulting from backfill deformation and non-linear soil response which lead to resonance effects within the backfill. Amplification factors of up to 1.5 were recorded in the reinforced backfill region.

- Changes in earth pressures recorded were due to a combination of increased earth pressures due to wall displacement and reduced earth pressures due to reduction of backfill density (backfill deformation) around the pressure cells. For the static earth pressures recorded, the outward displacement of the wall will also reduce earth pressures recorded as the backfill shifts towards the active state. Maximum seismic and static earth pressures were recorded at the wall toe of $1.5\sigma'_v$ and $0.9\sigma'_v$ (where $\sigma'_v$ is the vertical effective stress experienced at the location of the earth pressure cell).

- It was noted that reinforcement loads are dependent on the wall displacement, degree of rotation of the wall, reinforcement length and degree of confinement or magnitude of normal stress acting on the reinforcement layer.

- Highest reinforcement loads were recorded at the lowest reinforcement layer (deepest in the backfill) of 1.15kN/m for the representative wall model. A decrease in accumulated reinforcement load at failure for the lowest reinforcement layer is likely due to pullout failure of the reinforcement. Peak reinforcement loads of 0.7kN/m and 0.8kN/m were observed for the middle and top reinforcement layers with the latter experiencing gradual development of reinforcement load beyond 10mm of horizontal displacement only. An apparent load transfer from the lowest reinforcement layer to the middle and top reinforcement layers was noted at failure and is consequent to the pullout failure of the lowest reinforcement layer.

- A slight phase difference between the reinforcement load and wall displacement was observed. This is due to some initial resistance of the reinforcement layer to displacement (increase in reinforcement loads recorded) before some displacement is experienced by the reinforcement layer.
• In the initial excitation level, development of shear and contractive strains of 20% and 10% or greater along the reinforcement length is observed. Lower strain development was observed in the subsequent excitation levels. This initial strain development is likely due to the pull-out displacement of the reinforcement and ensuing contraction due to physical gaps between the geogrid and the backfill.

• The rotation of the wall resulted in subsequent development of inclined shear surfaces that extended from the backfill surface to the reinforcement ends. A progressive shift of strain development to lower shear surfaces with continual model excitation was observed.

• Failure of the model was initiated by failure plane development within the backfill at the excitation level prior to failure (at critical acceleration). Development of failure plane extending from the backfill surface to the lowest reinforcement layer and at the wall toe was observed at this stage.

• Connection of both failure planes through the extension of the failure planes along the R1 reinforcement layer is likely to have occurred at the excitation level before failure (critical acceleration level) forming a resultant failure plane similar to the two-wedge failure mechanism proposed by Horii et al. (1994).

• Development of this failure plane is likely associated with the onset of significant sliding displacement observed and is characterised by alternating zones of contraction and dilation of +/- 10% or greater along the failure plane. This indicates the occurrence of critical state shearing along the failure plane (constant volume shear).

6.2.2 Influence of wall parameters on GRS wall behaviour

The testing series of twelve wall models enabled conclusions to be made regarding the influence of wall parameters such as reinforcement length and layout, backfill density and backfill surcharging on the seismic behaviour of the GRS walls.

6.2.2.1 Influence of reinforcement length and layout

The staggered reinforcement wall model (increase in top two reinforcement lengths by 20%) was observed to result in higher wall stability (a reduction of wall top displacement by 10mm prior to failure) compared to the uniform reinforcement wall model (L/H=0.75). However, it is noted that the model excitation increments were too coarse to differentiate between the critical accelerations determined for the staggered and uniform reinforcement wall models. The longer top two reinforcement lengths were found to restrict the rotational mode of the wall resulting in greater sliding displacement observed. This is confirmed by the higher angle of rotation of the wall panel for the uniform wall models (9°) compared to the staggered reinforcement wall models (8.5°).

Amplification factors determined for the longer uniform reinforcement model (L/H=0.9) were found to be lower by a magnitude of 0.06 (at the 0.4g excitation level at 825mm elevation) due to a reduced model deformation compared to shorter uniform reinforcement wall models (L/H=0.75). For reinforcement loads, longer reinforcement lengths resulted in greater reinforcement loads recorded due to increased pullout resistance of the reinforcement layer. This was indicated by the reinforcement loads determined for the L/H = 0.9 wall models of 1.3kN/m compared to 1.0kN/m for the L/H = 0.75 wall models. In the staggered reinforcement models, a greater distribution of reinforcement load to the top two reinforcement layers was observed with reinforcement loads recorded (0.8kN/m at failure) greater than that recorded for the L/H = 0.9 model (0.6kN/m at failure) despite the same reinforcement lengths.
Strain development for the staggered reinforcement wall model is less than that observed for the uniform reinforcement model (L/H=0.75), indicating reduced internal backfill deformation brought about by greater stability. The longer top two reinforcement layers prevented the full development of a preliminary shear band at the middle reinforcement layer which would have contributed positively to the wall stability.

6.2.2.2 Influence of backfill density

The increase of backfill density from Dr = 50% to 85% was found to have a greater effect on wall stability than the increase in reinforcement length of 20% (from L/H of 0.75 to 0.9). A change in backfill density was found to have minimal impact on the mechanism of failure (overturning or sliding). The higher density wall model was observed to have lower amplification factors (1.1 compared to 1.25 in the unreinforced backfill). A linear reinforcement load profile was observed under static conditions indicating less wall movement was required to engage the reinforcement layers. This is due to greater confinement of the reinforcement for a dense backfill. For this reason, greater reinforcement loads were observed for the high density wall model (1.9kN/m compared to 1.1kN/m for the bottom reinforcement layer in the excitation level prior to failure).

Backfill strain development was observed to be significantly lower for the higher density wall model compared to its lower density counterpart with little to no radiating shear bands observed in the initial excitation level. This is likely due to the increased shear resistance of the higher density wall model. However, similar strain development between the wall models was observed along each reinforcement length which is likely because strain development along the reinforcement length is dependent on physical gaps between the reinforcement and backfill in the event of reinforcement displacement. Greater contractive strains were observed to develop in the lower density wall model which is in line with basic soil mechanics principles.

6.2.2.3 Influence of 3kPa surcharge

The application of backfill surcharge was found to initially increase the wall stability due to increased pullout resistance of the reinforcement but at higher excitation levels, the surcharge’s contribution to the wall destabilising inertial forces outweighed its positive contribution. The surcharge was observed to decrease the rotational component of displacement by increased pullout resistance of the reinforcement resulting in greater sliding displacement of the wall toe. However, for the longer uniform reinforcement model (L/H=0.9), due to a larger failure wedge, the surcharge had a larger contribution to the overturning force resulting in greater rotational displacement observed. Lower acceleration amplification factors were observed for the surcharged models as the surcharge acted to reduce the dynamic forces within the backfill, acting as a damper. An increase in the magnitude of reinforcement load developed was observed for surcharged models due to greater reinforcement confinement and increased destabilising forces acting on the backfill. However, minimal effect on the reinforcement load progression was observed.

The application of surcharge was observed to reduce strain development in the initial excitation levels but greater strain development was then observed in the later stages of model excitation. This is similar to its effect of wall stability and further demonstrates the shift in wall stability contribution of surcharging. The absence of a preliminary shear band observed for the top two reinforcement layers for the surcharged model indicates that the surcharge could have caused the position of initial
shear development to shift downwards to a lower shear surface. This could be due to increased pullout resistance of the top reinforcement layers due to surcharge application.

6.3 Model limitations

Despite the key conclusions derived from these experimental studies, there are some limitations to the applicability and accuracy of the results presented.

Firstly, one of the limitations of model similitude is that soil stresses and the corresponding soil-geogrid interactions at the prototype scale are unable to be replicated through the testing of the reduced-scale wall models under 1-g conditions. As a result, by possibly affecting the pullout failure strength and shear strength of the backfill, this could have affected the stability of the wall. However, it is unlikely that this would have affected the failure mechanisms observed as the wall model was only able to fail by overturning and sliding (Jackson, 2010). Additionally, the founding of the wall toe on the rigid box base (rigid boundary) instead of a foundation soil would also have affected the failure mechanisms observed as it would have excluded the possibility of a bearing capacity failure of the wall.

Due to the densification method used (excitation of the model with application of a compactor plate), it is possible that a non-uniform backfill density was achieved between the sand lifts. One of the difficulties encountered during experimentation is in estimating the target density for the first few sand lifts as further densification of these initial sand lifts is observed as model deposit construction progresses. This is due to the continual vibration of the sand lifts and increased vertical confining stress due to the continual construction of the model deposit.

Furthermore, due to only one type of facing tested, the failure mechanisms and progression of deformation observed in these wall models are only valid for GRS walls with a FHR wall panel. Other facing types would have resulted in different failure patterns observed such as bulging of the wall at mid-height observed for wrap-around wall models (Nova-Roessig and Sitar, 2006) and segmental (modular block) wall models (Matsuo et al., 1998).

6.4 Recommendations for future research

This thesis presents the second phase of a comprehensive study towards the development of seismic guidelines for GRS walls in specific geotechnical structures (embankments and bridge abutments) in New Zealand with the first phase of research completed by Jackson (2010). As a result, several key points are noted with regard to future research.

The use of the sand pluviation technique to construct the model deposit is proposed for further studies. This can be achieved at the University of Canterbury by attaching a detachable sand tray with pre-drilled holes to the sand deposition box and depositing the sand through this sand tray. Different target densities can be achieved by adjusting the size of the holes in sand tray and the height at which the sand is deposited. This methodology has been briefly trialled and was found to be successful in the later stages of research but the original model deposition technique involving excitation of the model was maintained to ensure model similarities throughout the research and also with the first phase of testing by Jackson (2010).

Due to the coarseness of the excitation level increments, the differences between the staggered reinforcement and uniform reinforcement models were not exhibited in the determination of the
critical acceleration of the wall models despite higher wall stability for the staggered reinforcement model (both wall models failed at the same excitation level). Therefore, smaller model excitation increments of 0.05g are recommended instead of 0.1g to produce a more refined progression of model excitation magnitude. Additionally, one of the limitations of the current research is that only one type of wall facing was tested resulting in the failure mechanisms and deformation progression observed being applicable to FHR wall panels only. Different facing options such as segmental wall facings (commonly used in New Zealand) are recommended for future wall models to assess their seismic performance.

In the current research, only three out of five reinforcement layers were instrumented with load cells to determine the reinforcement loads within each layer (due to the possibility that the load cells might adversely affect the wall behaviour). To enable a complete understanding of reinforcement loads achieved within the wall model, instrumentation of all reinforcement layers are recommended for future wall models. In addition, investigations to determine the wall toe loads developed during excitation are also recommended. This can be achieved by instrumenting the wall toe (resulting in a fixed wall toe). Previous studies (El-Emam and Bathurst, 2007) have shown that the wall toe has a significant influence on the wall’s sliding resistance, attracting 30-60% of the total horizontal earth force.

As part of the comprehensive study to develop NZ guidelines for GRS structures, the use of local soils as the backfill and its impact on seismic performance can be investigated in future research. However, note that key considerations will have to be made with regard to the similitude issues between prototype and model soil stresses as well as the soil-geogrid interaction. Additionally, models with different wall facing types should be tested in future research thus increasing the breadth of the research to incorporate typically utilised wall facings in New Zealand such as wrap-around wall facings and segmental (modular-block) facings. Lastly, the influence of loading on the wall panel itself to simulate abutment loading on the GRS wall is proposed for future research. This will investigate the performance of GRS walls when used as part of the bridge abutment foundation (similar to investigations by Tatsuoka (2008) on GRS integral bridges).

6.5 References


