4.7 Design implications

4.7.1 Discussion of acceleration amplification effects

Amplification of acceleration impacts on the magnitude of the lateral earth pressure on the wall (Steedman and Zeng 1990). It also contributes to an increase in the inertial force acting to destabilise the wall face and active zones of the wall. As evidenced in Section 4.4.6 above, this amplification occurs non-linearly up the wall, with the largest amplification occurring near the crest of the wall. This results in a larger inertial and seismic earth pressure component that occurs near the top of the wall.

FHWA (2001) incorporates an artificially modified design acceleration coefficient for use in seismic design using Equation 4-4 proposed by Segrestin and Bastick (1988). The non-linearity of amplification is ignored in favour of a uniform factor for application to the entire reinforced soil block.

\[ k_h = \frac{a_x}{g} \left( 1.45 - \frac{a_x}{g} \right) \]  

(4-4)

The equation is based on the average acceleration amplification over the wall height recorded during finite element analysis on two Reinforced Earth Walls on a hard rock foundation condition subjected to one earthquake time history of predominant frequency 8 Hz scaled to 0.1g, 0.2g and 0.4g peak base acceleration. Thus the results can hardly be generalised. However given that the boundary conditions of the model are highly idealised, Segrestin and Bastick (1988) notes that “any bias will be on the safe side.”

Based on this, the New Zealand Guidelines (Murashev 2003) use the similar Equation 4-5, though without justification for the reduced coefficient of 1.3.

\[ k_h = \frac{a_x}{g} \left( 1.3 - \frac{a_x}{g} \right) \]  

(4-5)

Figure 4-37 compares the amplification factor predicted from Equation 4-5 with the amplification exhibited in the vertical wall Tests-5 and 6 (L/H = 0.9 and 0.6) and the inclined Test-7 (L/H = 0.75; inclined 70°). Because the design acceleration is normally applied mid-height of the reinforced soil block, a linear interpolation has been used to determine the
amplification factor in the models at an elevation of 450 mm, as opposed to using the amplification factors recorded at wall top previously.

![RMS Amplification of acceleration measured near the wall top within the reinforced zone (Acc 4) and backfill (Acc 5) for Test-5 and 6 reinforced at L/H = 0.9 and 0.6 respectively.](image)

As shown, in practice Equation 4-5 results in amplification being considered for low acceleration levels, and a decrease in design acceleration at acceleration levels larger than 0.3g. Though it should be noted that for areas where the design acceleration is larger than 0.3g, the guidelines recommend a performance-based approach to investigate likely displacements given an earthquake occurs. Figure 4-37 also illustrates the following points:

- Amplification does not decrease with increasing base input acceleration as suggested by the NZ Guidelines, rather an opposite trend was observed and amplification, in general, increases with increasing base input acceleration. This was also confirmed by El-Emam and Bathurst (2007).

- The cut-off base input acceleration of 0.3g is too low to accommodate the full range of larger amplification that occurs at higher base input acceleration levels.
• The suggested maximum amplification factor of 1.3 is larger than the amplification factors of around 1.15 exhibited by the vertical models at mid-height of the wall. However, Section 4.6.4 plots a maximum amplification factor recorded at the wall top of around 1.43 for Test-6. Hence the 1.3 coefficient is conservative if the design acceleration is applied at mid-height.

In terms of design, these factors act to reduce design acceleration coefficients because GRS retaining walls have, in the past performed well. Thus they are a ‘fudge factor’ which seeks to reduce over-design.

Design also models the non-linear amplification as seen in Section 4.4.3 above, with a uniform acceleration applied to a ‘rigid’ reinforced soil block. Cai and Bathurst (1996) suggests that the discrepancy between actual non-linear amplification behaviour and design using an uniform acceleration can be somewhat rectified by the location of the resultant line of action for the dynamic earth pressure increment set to 2H/3. This rigid block assumption, used in deformation prediction models (Richards and Elms 1979) is violated with the observed acceleration amplification in the current and other tests (Nova-Roessig and Sitar 2006). Deformation observed during the current testing is discussed further in Chapter 5.

4.7.2 Performance-based design

The basis of performance-based design and prediction of deformation was discussed in Chapter 2. Performance based design has the ability to reduce the design acceleration coefficient by up to 50% in some cases in New Zealand, simply by allowing the occurrence of some movement in the event of an earthquake (Wood 2009).

Whilst there are a number of empirical and analytical models used to predict deformation (Cai and Bathurst 1996; Ling et al. 1997; Newmark 1965), the two most important parameters for the accuracy of any deformation prediction are:

• Determination of the most ‘appropriate’ ground motion for the simulation of the GRS wall design, and,

• An accurate measure of strength and mechanism of failure. Normally, this is a critical threshold acceleration.
While the first point is not the subject of this thesis, the current series of experiments highlight some issues with the second, that is, the correct identification of the critical acceleration. It was found that the critical acceleration governing ultimate failure occurred just prior to a significant increase in sliding displacement. Research has shown that Newmark sliding block theory is reasonably accurate for prediction of sliding displacement of GRS models (El-Emam and Bathurst 2004; Matsuo et al. 1998), and this method could be used to predict the final sliding response at failure.

However, the test results of these experiments (and others) demonstrate that deformation occurs even at low acceleration levels, and almost continuously. This deformation is predominantly by overturning of the wall. A Newmark-style analysis would not be able to quantify this element of deformation, as firstly the mechanism is not a sliding failure, and secondly, the method uses a constant threshold acceleration (Koseki et al. 2006).

Koseki et al. (2006) describes an attempt to simulate the gradual accumulation of displacement with increasing acceleration of reduced-scale model shake-table tests. The cyclic stress-strain properties of the model were simulated and displacement computed. This was compared to the displacement recorded in the experiments. Prior to ultimate failure which occurred at 750 gal, there was good agreement between the computed and experimental results. Thus accurate knowledge of the soil's properties may help in this method being used for designers to predict deformation prior to ultimate failure, and to compare this with the serviceability limit state criteria. While Newmark-type methods for block sliding could be used to compute sliding deformation at ultimate failure.

Chapter 5 investigates in detail the mechanisms of failure, and the development of deformation within the GRS models, to better aid in the prediction of deformation behaviour at low acceleration levels as discussed.

4.8 Summary

This chapter investigated in detail the seismic performance of reduced-scale GRS wall models with an FHR facing. The models were tested on University of Canterbury Shake-table. The chapter first introduced the test data obtained during one test, namely Test-6, a vertical wall reinforced with $L/H = 0.6$. Test-6 was selected because the construction methodology and testing procedure had been perfected allowing the most representative results of GRS behaviour under seismic loading to be obtained. The testing programme
consisted of excitation with a sinusoidal acceleration wave at 5 Hz frequency in stages of 10 second duration. The amplitude of acceleration was increased in 0.1g increments for each stage, with the first stage of testing at 0.1g. This input motion was selected to simplify interpretation of results and enable qualitative comparison with other research.

The deformation of Test-6 was illustrated step-by-step with photos captured at the end of each shaking step. This showed that Test-6, as representative of all other tests, failed predominantly by overturning. This involved the reinforced soil block rotating about the toe, coupled with multiple external shearing surfaces which formed within the backfill, with the first of these surfaces only just visible at the completion of the 0.2g shaking step. These shearing surfaces were inclined away from the reinforced soil block. Also evident is some minor sliding of the base of the facing panel along the rigid foundation, which occurred largely in the final shaking step of 0.5g during wall failure.

The deformation was described with the recorded acceleration and facing displacement time histories. Small deformation was recorded at the wall face at low accelerations up to some threshold acceleration, in this case, 0.4g shaking. Deformation consisted mostly by rotation, with only a small component of sliding observed. Failure occurred during the 0.5g shaking step, and was predominantly by overturning. During failure however, there was also a significant component of sliding.

In general, Test-6 demonstrated behaviour representative of all tests, with a characteristic bilinear displacement-acceleration relationship. This defines a critical acceleration value, below which only minor deformation occurs, and above which, significant deformation and failure is generated. Various definitions for critical acceleration proposed by different researchers were discussed, however, critical acceleration based on a sudden increase in the sliding component was selected as most appropriate to use for the current models. This is because failure for all models occurred concurrently with a significant increase in the sliding component of deformation. Reasons for why sliding might govern failure for the current model tests constructed with a FHR facing panel, are presented in Chapter 5. Critical acceleration can then be used for performance based design, and the prediction of displacements during an earthquake (however this was not discussed).

The parametric study consisted of seven tests designed to investigate the impact of reinforcement ratio L/H and wall inclination on seismic performance. The reinforcement ratio L/H was varied between L/H = 0.6, 0.75 and 0.9 for vertically faced walls (Tests-6, 1
and 5). One wall was inclined at 70° to the horizontal (Test-7). Three tests were repeated to validate the experimental method and also because of experimental error.

An increase in the L/H ratio from L/H = 0.6 to 0.75 to 0.9 was found to decrease the deformation pre-failure and increase the ultimate stability. For the range tested the acceleration level at which failure occurred increased from 0.5g, 0.6g to 0.7g respectively.

Similarly, wall inclination to 70° to the horizontal (Test-7 reinforced with L/H = 0.75) further decreased deformation pre-failure, with the deformation being somewhat similar to that recorded for the vertical wall, but reinforced at a larger L/H = 0.9 (Test-5). Sliding was small, yet as expected, larger than that observed for all vertical walls. Ultimate failure occurred during the 0.7g shaking step, and included a significant sliding component, again, larger than that observed for the vertical walls.

Amplification of acceleration is important because it increases the destabilising forces active on the wall. Amplification was found, for all tests, to be non-linear up the wall face, and to generally increase with increasing base input acceleration. A peak amplification of around 1.4 occurred during the final shaking step of 0.5g for Test-6. In the current NZ Design Guidelines (Murahsev, 2003) a maximum amplification of only 1.3 is specified, for low acceleration levels, and the trend used in design is of decreasing amplification with increasing base acceleration. The current GRS model experiments with an FHR panel facing, demonstrate that instead, an opposite trend of increasing amplification with increasing base acceleration was observed.

References


CHAPTER 5
PRE-FAILURE DEFORMATION OF GRS

5.1 Introduction

Post-earthquake case-studies provide evidence that GRS walls have in the past performed well: GRS walls in general demonstrated minimal to no damage in the 1995 Kobe and 1999 Taiwan earthquakes, when compared to conventional retaining walls which failed (Ling et al. 2001; 1996). While collapse of GRS walls has not been observed during earthquake shaking, the pre-failure structural performance is difficult to understand. Hence pre-failure deformations, that is, deformation because of base input acceleration shaking less than critical acceleration are of particular concern.

To investigate the performance of GRS walls pre-failure, coloured columns and horizontal lines of sand were layered against the transparent window in the model tests, and used to observe mechanisms of deformation within the reinforced soil block and retained backfill. The global mechanism of failure is first discussed in Section 5.2. Plots showing the development of these global failure mechanisms are inferred from photos taken at the end of each shaking step, and presented in Section 5.3. The observed failure surface angles are compared with those predicted by Mononobe-Okabe theory to determine its adequacy in assessing seismic behaviour.

In general, the sand markers were only able to show mechanisms of deformation that occurred with large shear strains. Geotechnical Particle Imaging Velocimetry (GeoPIV)
(White et al. 2003) is a non-invasive measurement technique and was used to accurately measure small strain fields within the reinforced soil block and backfill. This approach is discussed in Section 5.4, including its background and the validation procedures undertaken for its application in the current experiments.

GeoPIV was used to determine small-to-medium shear strains developed at low acceleration shaking, not detectable by the coloured sand markers. The interface between the reinforced soil block and the retained backfill is first analysed using this technique in Section 5.5. Section 5.6 then examines the strain field within the reinforced block itself, to test the design assumption that the reinforced block behaves as a rigid body.

The mechanisms of deformation was thus determined using two methods corresponding to deformation pre- and post-failure, using GeoPIV and sand markers, respectively. This information is combined to infer an idealised model of deformation for the GRS models with a FHR facing.

5.2 General description of observed failure mechanisms

Tests-6, 5 and 7, reinforced at $L/H = 0.6$, $0.9$ and $0.75$ and inclined at $90^\circ$, $90^\circ$ and $70^\circ$ to the horizontal respectively, were selected to study deformation mechanisms for different $L/H$ ratios and wall inclinations. These tests were also unaffected by testing errors and hence allow rigorous evaluation of seismic performance. In general, development of failure was similar for all tests and the step-by-step deformation of Test-6 shown in Section 4.1.3 describes this.

The mechanisms of ultimate failure for Tests-6, 5 and 7 are clearly illustrated in Figure 5-1 (a)–(c) respectively. All walls demonstrated predominantly rotation about the toe, which for the vertical walls resulted in overturning failure with some small components of sliding failure. The inclined wall (c) however, demonstrated a higher component of sliding than the vertical walls; as noted 40% of total deformation was sliding, compared to only 30% of total deformation for the vertical Test-5, and did not overturn (this was prevented by the experimental set-up).
Figure 5-1. Deformation visible after failure of Test-6 (a), Test-5 (b) and Test-7 (c) reinforced at L/H = 0.6, 0.9 and 0.75 (and inclined at 70° to the horizontal) respectively.

For all walls, the rotation was coupled with the formation of multiple failure surfaces within
the retained backfill. It can be seen that the failure wedge was largest for Tests-5 and 7 which failed at 0.7g and the angle of the lowest failure surface was shallower than compared to Test-6 which failed at 0.5g.

Similar observations of failure surfaces external to the reinforced zone have been reported in the literature (Fairless 1989; Howard et al. 1998; Koseki et al. 1998; Matsuo et al. 1998; Sabermahani et al. 2009; Sakaguchi 1996; Watanabe et al. 2003). It should be noted that no inclined failure surfaces formed within the reinforced soil block, except for at the toe of the wall (and this is explained in Section 5.3.4) in contrast to other research (El-Emam and Bathurst 2004; El-Emam et al. 2004; Lo Grasso et al. 2005; Sabermahani et al. 2009).

The occurrence of failure surfaces within the reinforced soil block can be attributed to the type of facing chosea (either: FHR panel, discrete segmental retaining, or wrap-around facing) and whether the reinforcement cover is sufficient. For instance, Watanabe et al. (2003) states that where the reinforcement is arranged properly, a failure plane will form with difficulty in the reinforced soil zone. Thus the failure mechanisms visible in the current model study, is consistent with what could be expected for FHR panel faced GRS models.

The vertical and horizontal marker lines in Figure 5-1 (a) – (c) show that the reinforced soil block also suffered some small deformation. This obviously included rotation, but also internal sliding along horizontal planes and settlement of the back of the reinforced soil block. Additionally, further shear surfaces can be seen to have formed at the toe of the reinforced soil block. These issues are discussed in Section 5.5 below.

5.3 Interpretation of deformation from images

5.3.1 Development of deformation

For all tests, the mode of failure included predominantly overturning and sliding. These external modes were accompanied by shear band formation in the backfill behind the reinforced soil block. The development of deformation differed between tests depending on the L/H ratio and wall inclination. Test-6 images viewed through the transparent sidewall and captured at the end of each shaking step are presented in Section 4.3.3, and images of Tests-5 and 7 are presented in Appendix C.

Discontinuities evident in the coloured sand markers were used to plot the progression of
failure within the backfill and reinforced soil block during testing. Figure 5-2, 5-3 and 5-4 (Tests-6, 5 and 7, respectively) plot the failure surfaces visible within the wall at selected acceleration levels.

With regard to identification of the shearing surfaces, it should be noted that video camera recordings enabled the global mechanism of deformation to be viewed and block movement showed the location of shearing surfaces prior to these becoming visible, after shaking. However, a failure surface was only recorded in Figure 5-2, 5-5 and 5-6 when a clear discontinuity became evident in the coloured sand columns or layers, which were around 4 mm thick, at the end of a shaking step. Thus, displacement of around 2-3 mm had to occur for the failure surface to be noted. This somewhat crudeness in measurement is rectified by use of GeoPIV analysis as discussed in Section 5.5.

The location of the reinforcement post-failure was determined by excavation; at shaking steps pre-failure the location of the reinforcement was assumed based on the location of the horizontal layers of coloured sand placed at heights corresponding to the reinforcement (see Section 3.7.3). Excavation revealed that, in general, the coloured sand layer followed the reinforcement closely and hence this assumption was valid.
Figure 5.2. Progression of deformation within the backfill in Test-6 reinforced L/H = 0.6 at the completion of: a) 0.3g, b) 0.4g and c) 0.5g (failure).
Figure 5-3. Progression of deformation within the backfill in Test-5 reinforced L/H = 0.9 at the completion of: a) 0.3g, b) 0.5g, c) 0.6g, and d) 0.7g (failure).
Figure 5-4. Progression of deformation within the backfill in Test-7 reinforced L/H = 0.75 with a facing inclination of 70° at the completion of: a) 0.4g, b) 0.6g and c) 0.7g (failure).

Figure 5-2, 5-3 and 5-4 show the progression of deformation for walls reinforced by L/H = 0.6, 0.9 and 0.75, with the latter wall also inclined at 70° to the horizontal (Tests-6, 5 and 7 respectively). For all tests, multiple failure surfaces at varying inclinations and elevations formed at different acceleration levels. The failure surfaces were accompanied by overturning of the wall face which allowed sliding of an active wedge along the failure surface into the back of the reinforced soil zone to occur. A number of general comments are made with reference to the above plots:
For Tests-6 and 5, the first and highest shearing surface became visible during 0.3g shaking step. For Test-7, the first and highest shearing surface became visible during the 0.4g shaking step.

Shaking at higher base acceleration levels either propagated the existing shearing surface (by sliding of the active wedge along the shearing surface), and/or generated a deeper shearing surface. For the vertical walls, this was accompanied by a vertical shearing surface that appeared to propagate downwards from the wall crest and to trace the back of the reinforced soil block.

Except for Test-7, the angles of subsequent lower shearing surfaces were similar (within 5°) to that of the previously formed failure surface’s initial angle.

Failure surfaces were reasonably planar, especially at lower elevations. The surface became curved near the top of the wall. Slight irregularities in the inclination of the failure surface were possibly due to local strength differences, progressive deformation, the mode of deformation (rotation), and/or measurement error.

Apart from near the back of the reinforced soil zone, the horizontal reinforcement prevented an inclined failure surface from compromising the reinforced soil block.

As the failure wedge displaced and deformed, failure surfaces higher up within the wedge became visible and were recorded.

As the wedge displaced downwards, high settlements occurred behind the reinforced soil block, and the ends of reinforcement were “dragged” downwards. Sabermahani et al. (2009) noted similar down-drag forces acting on reinforcement tails near the back of the reinforced block.

Except for Test-7, the angles of previously formed failure surfaces were all reduced (became shallower) as the active wedge slid down the failure surface into the reinforced soil block. This possibly indicates that the active wedge rotated clockwise during subsequent shaking steps, as the wall face rotated towards the vertical.

Specific impacts on deformation due to the L/H ratio and wall inclination are discussed subsequently.
5.3.2 Effect of L/H ratio on deformation

In general, the deformation patterns for both vertical wall tests was similar even though there was a 50% increase in reinforcement length from Test-6 to Test-5. As noted previously, this increase in reinforcement generated greater stability (a larger critical acceleration). The following comments are made with regard to the influence of L/H ratio on failure surface formation.

- The angles of the failure surfaces formed in Test-6 (L/H = 0.6) were steeper than the angle of the failure surfaces formed in Test-5 (L/H = 0.9): Approximately 55° compared to 42°.

- Test-6 (L/H = 0.6) reinforced soil block exhibited less downward deformation of the reinforcement ends. This perhaps indicates that the L/H = 0.6 reinforced soil block acted in a more rigid manner than the L/H = 0.9 reinforced soil block.

5.3.3 Effect of wall face inclination on deformation

Figure 5-4 plots the progression of deformation for Test-7 with a 70° inclined wall face reinforced at L/H = 0.75. Up until 0.4g shaking, deformation was primarily restricted to rotation of the wall face and no noticeable failure surfaces were formed within the backfill. By the completion of 0.4g shaking step, one failure surface was recorded, at an inclination of 35°. This angle is shallower again than the vertical wall, reinforced at a longer L/H = 0.9. During the 0.7g shaking step, the deepest failure surface formed and was inclined at 41°, similar in value to that which formed at 0.7g for the vertical wall L/H = 0.9. Further failure surfaces became visible above this as the active wedge deformed whilst it slipped down and into the back of the reinforced block.

Upon comparison with Figure 5-3 and Figure 5-4, Test-7 exhibited fewer failure surfaces prior to failure at 0.7g, illustrating an increased stability, due to the reduced inclination of the wall.

5.3.4 Progressive failure

In limit equilibrium methods, the active wedge is assumed to mobilise peak soil strength at the same time at all points along a potential failure surface. In reality however, progressive failure phenomena means that along a potential failure surface, different strengths according
to location on the stress-strain curve will be mobilised. Thus failure is initiated along the surface when the peak strength is reached at some point and resistance drops (with continued strain) to some residual strength value on the stress-strain curve. This in turn causes a redistribution of stresses along the potential failure surface, and possibly failure at other points along the potential failure surface. Hence peak strength is not mobilised at the same time during shaking and the failure surface is propagated accordingly.

Figure 5-2 and Figure 5-3 provide evidence of progressive failure along each failure surface. During increased base input acceleration, failure surfaces were initiated within the backfill. The failure surface only became visible once sufficient sliding displacement (shear strain) of the active wedge occurred for post-peak reduction of the shear resistance to residual strength values. This redistributed the shear stresses along the potential failure surface. At continued shaking, or higher acceleration shaking, the failure surface was propagated down towards the back of the reinforced soil block.

Watanabe et al. (2003) reported progressive failure within the backfill of conventional gravity-type retaining wall models. In these tests, a failure surface formed which extended from the wall crest to the heel of the wall. Upon application of larger acceleration amplitude shaking a second potential failure surface exploited the localised weakness at the wall heel and initiated a second failure surface up into the backfill inclined at a shallower angle.

In the absence of a reinforcement layer along the solid foundation, the heel of the wall in the current experiments is, in effect, similar to a conventional retaining wall. Figure 5-5 shows the two failure surfaces formed at the wall heel during the final shaking step of 0.5g for Test-6.
Figure 5-5 shows the two failure surfaces intersect the bottom layer of reinforcement, R1 (see Section 3.4.8 for reinforcement numbering). In accordance with similar observations made by Watanabe et al. (2003) noted above, it is likely that during the final shaking step of 0.5g, the steeper failure surface formed first, then, as the wall slid along the bottom layer of reinforcement, the second failure surface formed possibly exploiting the weak region at the toe of the wall (due to strain softening along the previously formed failure surface).

5.3.5 Comparison of angle of failure surfaces observed with those predicted by Mononobe-Okabe theory

Mononobe-Okabe (MO) theory is commonly used to calculate psuedo-static earth pressures developed within geotechnical structures under seismic loading, as described in Section 2.2.3. Further, the theory also allows the prediction of the angle of the failure plane which forms the active wedge. As the design acceleration coefficient increases, the seismic component of total earth pressure, $K_{AE}$, also increases, and this indicates that the active wedge has increased in volume, and the failure surface becomes shallower.

It has been noted that at large design acceleration coefficients, MO theory is conservative (Murashev 2003), therefore it is important to compare the MO theory predictions of the failure surface angle with that observed in the current tests. The parameters used to calculate the angle of the failure surface for each shaking step were assumed as: Internal angle of
friction of Albany sand, 33°; Backfill slope, 0°; Wall inclination to the horizontal, 90°. The angle of friction between the face and soil was assumed equal to 3/4 of the internal angle of friction of Albany sand (Koseki et al. 1998) and hence was assumed equal to 25°. Table 5-1 shows the predicted angles of the potential failure surface, and compares these with those observed. Figure 5.6 shows the definition of the angle of the failure surface predicted and observed.

![Figure 5-6. Schematic diagram showing the definition of the failure surface angle, α_{AE}, for the MO prediction and that observed.](image)

Table 5-1. Comparison of angles of failure surfaces at various shaking levels predicted by Mononobe-Okabe theory and observed for Tests-6, 5, and 7 (reinforced by L/H = 0.6, 0.9 and 0.75, the latter wall inclined at 70° to the horizontal, respectively).

<table>
<thead>
<tr>
<th>Shaking step</th>
<th>MO predicted α_{AE} (vertical walls)</th>
<th>Observed α_{AE}</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Test-6</td>
<td>Test-5</td>
<td>Test-7</td>
<td></td>
</tr>
<tr>
<td>0 (static)</td>
<td>57°</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>0.3g</td>
<td>39°</td>
<td>63°</td>
<td>47°</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>0.4g</td>
<td>31°</td>
<td>63°</td>
<td>42°</td>
<td>35°</td>
<td></td>
</tr>
<tr>
<td>0.5g</td>
<td>22°</td>
<td>53°</td>
<td>42°</td>
<td>35°</td>
<td></td>
</tr>
<tr>
<td>0.6g</td>
<td>11°</td>
<td>-</td>
<td>41°</td>
<td>35°</td>
<td></td>
</tr>
<tr>
<td>0.7g</td>
<td>indeterminate</td>
<td>-</td>
<td>42°</td>
<td>41°</td>
<td></td>
</tr>
</tbody>
</table>

There are three important points to be made with the derivation of Table 5-1. Firstly, whilst wall inclination can be expected to alter the earth pressure within the wall, MO theory is indeterminate for a slope of 70° to the horizontal given the other parameter values just noted.
Hence, the observed angles for Test-7 are compared with that predicted for a vertical wall. Secondly, because the failure surfaces were observed to be somewhat curved, (becoming shallower at higher elevations within the wall) the angles listed in Table 5-1 were taken from the lower sections of the wall where visible. Finally, the angles recorded were made by the lowest failure surface visible within the wall.

Table 5-1 shows that MO theory predicts shallow failure surfaces at high design accelerations, with an angle of 11° predicted for the 0.6g shaking step. At 0.7g, MO theory is indeterminate, and this corresponds to the condition where the entire wall is assumed unstable and limit equilibrium does not exist. Upon comparison with the angles observed, MO theory can be considered highly conservative, predicting potential failure surfaces far shallower than those observed.

A number of reasons for the disparity between that predicted and that observed are listed below.

- The mechanism of failure between that predicted with the MO theory and observed are different. The MO assumes a single failure wedge, whereas the GRS models instead displayed the two-wedge failure mechanism, as proposed by Horii et al. (1994).

- Progressive failure is shown to occur, contrary to the limit equilibrium assumptions of MO theory (as discussed in Section 5.3.4).

- The MO prediction is sensitive to the soil-wall face interface friction value chosen.

- Multiple failure surfaces develop within the backfill of all models.

Similar differences between that predicted using MO theory, and that observed have also been made by Koseki et al. (1998) and El-Emam and Bathurst (2004) on shaking-table tests of GRS models. The MO predictions of the former were conservative (i.e. shallower), and the latter were un-conservative (i.e. steeper). This indicates the ineffectiveness of MO theory for application to GRS walls.

5.3.6 Comment on sidewall friction

Exact values for the inclination of failure surfaces should be interpreted with caution as the boundary conditions of the acrylic sidewall are not perfectly non-frictional. Watanabe et al.
(2003) investigated the effect of sidewall friction on the formation of failure surfaces during a series of shake-table experiments on conventional and reinforced soil retaining wall models in a strong-box 600 mm wide. In this test, 3 accelerometers were located across the width of the wall at the same elevation. Additionally, a thin brittle wire was run along the box centerline from the wall face to the back wall. The wire was electrified for the purposes of determining the timing of formation of failure surfaces along the centerline: a failure surface would cause the wire to break with a measurable drop in voltage across the circuit.

It was found that there was negligible difference between the accelerations recorded at the sides and centerline of the wall. Further, formation time of the failure surface viewed from the side wall via high-speed camera was the same as that which occurred along the box centre line determined from the brittle electrified wire. However, upon excavation along the box centerline, the angle of the failure surface was found to be 6° shallower from that viewed through the Perspex sidewall.

Considering that no special measures were taken to reduce sidewall friction in the current experimental setup, it is reasonable to presume that the failure surface angles observed through the transparent sidewall would indeed have been shifted by some angle from that at the box centerline.

5.4 Geotechnical Particle Imaging Velocimetry

During testing, high-speed images of two regions: the reinforcement soil block, and the interface region between the reinforced soil block and the retained backfill, were captured for each shaking step. Post-processing of the image data was then undertaken using GeoPIV, to calculate the displacement and strain fields within these regions.

In Section 5.4.1 the background to this technique is first provided followed by possible errors inherent in the process (Section 5.4.2). Section 5.4.3 discusses the regions selected for in-depth GeoPIV analysis. The initial tests undertaken to validate the use of GeoPIV for the current experimental programme are described in Sections 5.4.4 and 5.4.5.

5.4.1 Background of GeoPIV technique

The Particle Image Velocimetry (PIV) technique was originally adapted for experimental fluid dynamics by Adrian (1991) and more recently has been used in varying fields such as
the accurate quantification of particle velocity Fritz et al. (2004); and to measure wall shear-rate in blood pumps (Kim et al. 2004). For the purposes of non-intrusive measurement in geotechnical engineering applications, PIV techniques have been developed to measure foundation displacements under load (White et al. 2003); and to identify displacements and shear strains within retaining wall models tested in a centrifuge or on a shake-table (Zornberg et al. 2003; Watanabe et al. 2005).

PIV is a generic term which covers many different image processing techniques (it is also referred to as a “Digital Image Correlation” or DIC). In general the PIV process utilizes high speed cameras, lenses and accurate timing devices to capture image information at a predetermined rate for analysis with PIV software. The GeoPIV software for geotechnical engineering applications developed and described by White et al. (2003) is the particular technique used in the current experiments.

PIV enables image texture, via the different colouration of soil particles, to be identified and displacement measured from successive high speed images (frames) over an accurately known time difference. The image is first divided into a mesh of ‘patches’, sized a certain number of pixels, and the texture of each patch (pattern of light and dark) is recorded. The texture of each patch can then be traced within successive images, to determine patch movement. In contrast to the image measurement procedures used by Zornberg et al. (2003) and Watanabe et al. (2005), GeoPIV does not require sand markers to be placed into the soil; rather GeoPIV utilizes a soil’s texture within an image and thus avoids the need for intrusive calibration devices within the experimental specimen (White et al. 2003).

The texture must be sufficient for the algorithm to recognize discrete patches accurately, otherwise particles need to be “seeded” into the specimen (Bolinder, 1999). “Seeding” involves the use of more visible particles that alter the contrast of the material and allow the texture to be more easily recognised by the GeoPIV software during post processing.

In the current experiments, “Seeding” of small pebbles into the sand layered closest to the side wall window was trialed in Test-2, but the large quantity necessary made this difficult to achieve. Further, “seeding” was not deemed necessary as the Albany Sand and lighting conditions enabled sufficient texture to be visible for analysis with the GeoPIV software. However, it was found that texture was increased with the addition of horizontal and vertical layers of dyed sand, which were used to better visualise failure planes by eye as shown in
GeoPIV was used to track the texture within ‘patches’ of successive images by matching textures with the highest correlation within the second image taken at a known time interval; this indicates the vector direction of each patch between images. GeoPIV then determines the exact location of the correlation peak through a bi-cubic interpolation of surrounding correlation values which can generate sub-pixel accuracy (Bolinder 1999). The general principles of GeoPIV are summarised in Figure 5-7.

![Diagram of GeoPIV principles](image)

**Figure 5-7. Principles of GeoPIV (from White and Take (2002)).**

The directions and magnitudes of patch movement as seen in the image captured by the high-speed camera are modified from “image-space” (in pixels) to object-space (measured for instance in mm) by correlation with a grid of fixed points on the surface of the transparent acrylic window built into the box. This process is termed calibration and is discussed in Section 5.4.2 and 5.5.1.

From the PIV data collection process, essential information such as location of failure planes, the timing of formation, and the magnitude of local strains can be obtained.
5.4.2 Discussion of error in GeoPIV analysis

The GeoPIV measurement system involves two types of error generation: the precision, and the accuracy of the system. GeoPIV constructs the vector displacement field in image space; the difference between the true movement of patches and that recorded is the precision of the GeoPIV system and data collection methods (camera). Accuracy is dependant on data acquisition methods to transform the displacement field generated in image space to that within real-space. This process is known as image calibration and is discussed in Section 5.5.1.

Both sources of error must be measured and quantified to determine the validity of the system. White et al. (2003) evaluated the precision of the GeoPIV programme in a series of computer simulations and physical experiments on a translating bed of non-deforming Dog’s Bay Sand. For the computer simulations an artificial “random” image was translated artificially. For the physical tests, micro-metered incremental translations were applied to a body of sand and images recorded via a rigid-fixed camera. GeoPIV displacement analysis was conducted with varied patch size and compared with the displacements known by the micro-meter instrumentation.

The experiments showed that precision is a strong function of patch size (i.e. the larger the patch the greater the precision), and a weak function of image content (i.e. patches of low texture are more difficult to locate precisely than high texture patches).

For instance, the patch sizes ranged from 5 by 5 to 50 by 50 pixels. For the sand, such an increase in patch size resulted in an increase in the precision of GeoPIV tracking by a factor of 18. For the sand with patches sized 50 by 50, the standard error per pixel was approximately 0.01 (as a fraction of pixels).

The image content is described in terms of its spatial brightness frequency. The spatial brightness of a soil is lower than an artificially created image. This is because for a soil, the brightness of multiple pixels is not necessarily independent, i.e. the pixels could be on the same grain of sand and hence be the same brightness. For instance, a given patch size of 32 by 32 pixels on a random image (where 2 by 2 pixel blocks have a random brightness independent of their neighbours) was tracked using GeoPIV approximately 7 times more precisely than the sand.
Figure 5-8 shows a conservative upper bound on the precision error for a mesh of patches (each sized L by L pixels) and the number of patches (measurement point array) within the mesh. Precision is expressed as a fraction of the width of the Field of View (FOV) which is the width of the image under inspection.

![Figure 5-8. GeoPIV precision and measurement array size vs camera CCD resolution (from White and Take (2002)).](image)

Thus a compromise is needed in deciding the optimum patch size: Larger patches can provide improved precision, though in regions of high strain gradients, smaller patches (and hence more measurement points) can reveal greater detail.

The second type of error is generated during calibration of displacements measured in uv-space (pixel) to displacements measured in xy-space (mm) of the physical model. Either a single scale factor is used to convert between pixel and object coordinates or photogrammetry is used. The former is rarely accurate in short focal length digital photography due to spatial variation in image scale caused by: ensuring the camera and object planes are coplanar, radial and tangential lens distortion, refraction through the viewing window and pixel non-squareness (minor) (White and Take 2002).
Because of the above-mentioned difficulties, an accurate photogrammetric transformation from uv-space to xy-space is needed. The transformation first determines the spatial variation in scale caused by the abovementioned variables by tracking the location of a set of dots of which the xy-space coordinates are accurately known. A Mylar sheet is typically used for this (White et al. 2003). These accurately known dots are then compared with the reference points located on the inside of the box window (shown below in Figure 5-11) which are later used to calibrate the soil patch displacements from uv-space, to object displacements in xy-space.

Hence the accuracy of the transformed displacements is the geometric summation of the precision of first tracking the reference points, and secondly, the precision of tracking the soil patches with GeoPIV. Again, both these precision errors can be conservatively estimated from Figure 5-8. The precision and accuracy related errors are summed over the course of a GeoPIV analysis of several images and correspond to a random walk of error generation. This random walk determines the error of a GeoPIV analysis.

For the current application, in general each patch was traced between two successive images with a standard error per pixel of 0.1. The FOV (field of view) for the reinforced soil block window shown in Figure 5-11 was 640 pixels. Hence each patch could be traced with a precision error of 0.015%.

The GeoPIV program has an in-built “Leap frog” function, which is used to reduce the accumulation of random walk error. The assigned Leap frog value, indicates how many images will be compared before the initial image is updated. For instance, if Leapfrog is set to 1, then images 1 and 2, 2 and 3, 3 and 4 will be compared. However the errors will be summed in a random walk fashion. If Leap frog is set higher to 3, for example, then images 1 and 2, 1 and 3, 1 and 4 are compared, before updating the initial image and comparing 4 and 5, 4 and 6 etc. This reduces the random walk error, however because images are being compared over larger time frames, patches may deform, and this makes tracking each patch difficult and can result in a larger number of wild vectors.

In the current experiments, Leap frog was used depending on the analysis being run. Where residual strains were investigated, the images compared were pre-selected to occur at cycle peaks or the end of shaking steps and this created a similar leap frog effect. In analyses’ where the cyclic nature of strain was investigated, and hence many images in succession were analysed, leap frog was used to reduce the random walk error during the analysis.
5.4.3 Selected regions within the wall for GeoPIV analysis

Camera resolution and memory limited the window size for investigation. A large window may result in insufficient texture, as at larger distances, the sand begins to look homogenous. Additionally, a larger window requires a larger memory built into the camera. In high-speed photography, camera memory is often a limiting factor, and this acts to limit the size of the ‘windows’ under investigation. The camera properties are presented in Section 3.6.3.

Two areas were selected initially as “windows” to focus on during testing and are shown in Figure 5-9. Window C1 captured data of the front region of the reinforced zone along reinforcement layer R4 at 600 mm elevation. This region encompassed the soil layer 75 mm vertically above and below the reinforcement, and its horizontal location was varied between tests. Reinforcement layer R4 was selected because design codes typically cite the location of peak lateral dynamic earth pressure at 60% of the wall height (0.6 x 900 = 540 vertical height).

![Figure 5-9. Location of “windows” in Test-6 (identified by dashed white lines), used for image capture by cameras C1 and C2. The images captured of these windows were then analysed using GeoPIV.](image)

Window C2 was centred on the interface region between the reinforced block and the retained backfill. The area encompassed 100 mm horizontally either side of the interface and from 75 mm below reinforcement layer, R2, to 75 mm above reinforcement layer, R4.
5.4.4 Validation of GeoPIV for the current application

Validation of the GeoPIV programme was performed during its development by White and Take (2003). The systems accuracy and precision was discussed in Section 5.4.2. Nevertheless, it is important to ensure that the results generated by the programme are reliable for its current use.

To check the image acquisition system and to validate the calibration process for the current experiments, a simple sinusoidal excitation was applied to one wall prior to un-bracing of the wall, and recorded with the three cameras as described above. The displacement time history of the shake-table motion is shown in Figure 5-10.

![Displacement-time history of the shake-table](image)

Figure 5-10. Displacement-time history of the shake-table used for PIV validation test. NB: The circled points are images taken for validation of GeoPIV.

As in the testing protocol, camera recording was “triggered” by a 3 volt switch synchronised with the shake-table control system. This ensured that an accurate time zero was established such that the correct image, corresponding to a displacement peak for instance, could be determined.
In order to determine the performance of GeoPIV for the current investigation, its ability to record displacement over the first half cycle of a 0.1g shaking motion was investigated. A photo of the reinforced soil block and retained backfill interface region taken at point 1 of the shake-table displacement-time history (Figure 5-10) is shown in Figure 5-11 (a). A mesh of patches of 32 by 32 pixels was created for the region shown in Figure 5-11 (b). The GeoPIV programme was then used to track the displacement of each patch between the 3 images corresponding to points 1, 2, and 3 of Figure 5-10.

The displacement vector fields between points 1 and 2, and 2 and 3 are shown in Figure 5-12 (a) and (b). Each vector is plotted in dimensions of pixels in u-v space and has been scaled up 25 times to better view the patch displacement. The vector field traces soil patch displacement field of the wall corresponding to a 1 mm shake-table displacement. As can be seen, there are no “wild” vectors, and this indicates that the soil’s texture is sufficient for tracking.
Figure 5-12. Displacement vector field of interface region in u-v space (pixels). Vectors plotted between:
(a) Time history points 1 and 2, and (b) points 2 and 3.

The above displacement plots tracked patches of soil as it moved with the displacement of the shake-table. Thus the displacement plotted in Figure 5-12 includes the displacement of the shake-table as well. The displacement of the soil relative to the shake-table is determined through the calibration process. In this, the displacement of the reference dots (Figure 5-11(c)) is traced through the image sequence and removed from the displacement of the soil patches shown in Figure 5-11 (b). Thus the relative soil patch displacement can then be plotted in real xy-space (mm) and is shown in Figure 5-13 (a) and (b). Because the wall was braced, and the shake-table motion was gentle, the relative displacement of the soil, as expected, approximates zero.
Figure 5-13. Calibrated displacement vector field of interface region in x-y space (mm). Vectors plotted between: (a) Time history points 1 and 2, and (b) points 2 and 3.

The above validation process confirms that:

- Patches of Albany sand had sufficient texture and were able to be tracked through a simple sequence of images;

- The tracking process identified simple cyclic displacement;

- The calibration process was able to determine relative soil displacement through the removal of shake-table displacement.

5.4.5 Determination of appropriate patch size

As noted above, the precision is dependent on the patch size of the mesh. An image sequence for one test was analysed using three patch sizes to determine the most appropriate patch size for subsequent analysis. The patches are square and measured in pixels sized 16 by 16 pixels, 32 by 32 pixels, and 64 by 64 pixels. The calibrated vector plots (i.e. in xy-space) for each analysis is shown in Figure 5-14.
Figure 5-14. Displacement vector plot for comparison of patch size for GeoPIV analysis: a) 16 by 16 pixels, b) 32 by 32 pixels, c) 64 by 64 pixels. Each vector has been scaled up by a factor of 3.

As the patch size increases the accuracy is improved; there is more information in each patch to track through the images. At small patch sizes, an un-manageable number of “wild” vectors are generated in the GeoPIV analysis. In general, wild vectors are removed by manual inspection, however Figure 5-14 (a) shows that too many wild vectors are generated to consider this feasible. However, while large patch sizes generate a smaller number of wild vectors as in Figure 5-14 (c), the displacement field is unnecessarily crude and displacement information is lost. A compromise is needed and it was deemed appropriate to size patches 32 by 32 pixels for subsequent analysis. As shown in Figure 5-14 (b), some post-processing to remove the wild vectors is necessary, however the burden is not large.
5.5  Deformation of the reinforced soil block – retained backfill interface prior to failure

5.5.1  Deformation of the Test-6 model using GeoPIV

GeoPIV was used to analyse the residual deformation of the interface region of Test-6. The region ("window") under analysis was shown in Section 5.4.3. Residual deformation is analysed as deformation that occurred between the start of the test and the end of each shaking step. The analysis does not consider changes in deformation which occur within a shaking step. Rather, images are selected from the start of the test, and at the end of each shaking step, and the deformation is analysed using GeoPIV. This calculates the shear strain accumulated within the reinforced soil block-retained soil interface, from the initial position until wall collapse.

As they were typical of the test results, GeoPIV results from Test-6 are used in this section to demonstrate the GeoPIV analysis procedure. First, the displacement of each patch was determined and shown as a vector field plot in Figure 5-15. This shows the displacement accumulated from the start of the test to the end of the shaking steps 0.1g – 0.4g, as indicated. The final shaking step of 0.5g was not plotted due to the unreliable results that are generated. This was because the mechanism of failure during the final shaking step (in this case 0.5g) involved a failure surface developing below the window used for GeoPIV. The resultant wedge encompasses the entire window under consideration and the soil within translated quickly and outside of the window and image. Thus GeoPIV was unable to track patches reliably during the last shaking step, however, this stage of deformation is clearly visible with the naked eye.
Figure 5-15. Displacement plot of interface region for shaking steps 0.1g, 0.2g, 0.3g and 0.4g. (Note vectors have been scaled by 3). The plotting region is the extent of the viewing window. Vector circled for inspection below.

Figure 5-15 shows the progression of accumulated displacements during testing of Test-6 within the test window, located from 450 to 650 mm in the horizontal x-direction, and 250 to 650 mm in the vertical y-direction. Minimal displacement occurred during the 0.1g shaking step; a steady increase in soil displacement is, however, visible with increasing base input acceleration. The location of a shearing surface is visible after the 0.2g shaking step that extends from the lower left hand corner to mid-height along the right side of the window (x, y coordinates: (500, 275) to (600, 425)).

Significant movement is visible above the shearing surface and can be associated with overturning of the wall face allowing the active wedge to slide downwards and into the back of the reinforced soil block.

The length of a displacement vector, circled in Figure 5-15 (d), and located midway along the shear surface after 0.4g shaking step is 12.9 mm (note that each vector in the above plot has been scaled by 3). The length of an inclined vector on the lower side of the shear surface is 3.7 mm. Hence the differential displacement along the shear surface is 9.2 mm.

Inspection of the vector plot (Figure 5-15) shows that up to 16 mm lateral displacement
occurred at the top of the interface region, corresponding to 25 mm lateral displacement recorded with the displacement transducers at the same elevation at the wall face. Similarly, lateral displacement below the shear surface of approximately 2.0 mm by completion of the 0.4g shaking step compares reasonably well with the sliding displacement of 3.6 mm recorded at the base of the wall face. These results demonstrate a similar order-of-magnitude, however it should be noted that values recorded at the wall face, and at the reinforced soil block - retained backfill interface will never be the same. This is due to the reinforced soil block acting non-rigidly, and hence displacement recorded at the wall face includes deformation of the reinforced soil block.

Also of interest in Figure 5-15, is a near-vertical line defined by a sudden change in the vector direction from inclined to near horizontal. It appears that the line occurs roughly at the interface between the reinforced soil block and retained backfill and is located at roughly x = 520 mm. Note that due to some movement of the wall face, the location of the reinforcement ends has changed from its initial location at x = 540 mm.

In order to better understand the relative deformation of the soil, strains can be calculated using GeoPIV. To do this, GeoPIV creates a triangular mesh of elements which link the centre of patches with its neighbouring patches. Relative patch displacement causes extension or compression of these elements; this is used to calculate various components of strain that include the maximum shear strain, shear strain, linear strain, or volumetric strain. Figure 5-16 shows the triangular element mesh (in uv-space) after “wild” vectors and associated patches within the window were removed during the cleaning process.
Figure 5-16. Triangular element mesh for strain calculation plotted in xy coordinates. Note that because some "wild" vectors were evident in the displacement plots, the associated patches have been removed from the element mesh.

Equation 5-1 presents the maximum shear strain calculated from the relative extension and compression of elements linking adjacent patches.

\[ \gamma = \sqrt{(\varepsilon_x - \varepsilon_y)^2 + \gamma_{xy}^2} \]  \hspace{1cm} (5-1)

Where \( \gamma \) is maximum shear strain, \( \varepsilon_x \) and \( \varepsilon_y \) are linear strains in the x and y directions respectively, and \( \gamma_{xy} \) is the shear strain in the x-y plane. The maximum shear strain, \( \gamma \), is plotted primarily for the current purposes, however its various components are used to further scrutinise the modes of deformation where necessary.

Figure 5-17 plots the progression of residual maximum shear strain within the soil window accumulated by the completion of 0.1g, 0.2g, 0.3g, and 0.4g shaking steps for Test-6 reinforced at L/H = 0.6. Colour contours are used to denote percentage values of localised strain.
Figure 5-17. Residual shear strain of the reinforced/retained backfill interface of Test-6 reinforced with L/H = 0.6 and accumulated by the completion of: (a) 0.1g, (b) 0.2g, (c) 0.3g and (d) 0.4g shaking steps.

Figure 5-17 shows localised maximum shear strains are evident by the completion of the 0.2g shaking step, with a shear surface formed between the second reinforcement layer from the bottom (R2), and strains beginning to accumulate at the ends of the third and fourth layers of reinforcement (R3 and R4), originally located at 540 mm horizontally and at 300, 450 and 600 mm elevations. The location of each layer of reinforcement is shown with a dashed line.
in Figure 5-17 (a). Further shaking at 0.3g and 0.4g contributes to increasing shear strains along well-defined shear surfaces.

Figure 5-17 (b) shows the dominance of an inclined failure surface that intersects with reinforcement R2. Other higher shear surfaces which appear to intersect with reinforcement layers R3 and R4 are visible, but these have not been as well developed. This would appear to be inconsistent with the progressive failure model which prescribes the successive formation of deeper failure planes. The shear strain accumulated by the completion of 0.1g shaking step is replotted with a finer scale to better visualise this initial development of deformation.

![Figure 5-18](image)

Figure 5-18. Residual shear strain accumulated by the completion of 0.1g (replotted at a finer scale for better visualisation) of the reinforced/retained backfill interface of Test-6 reinforced with L/H = 0.6.

Figure 5-18 shows that, by the completion of 0.1g shaking step, deformation did indeed occur, and that this was confined to near the ends of reinforcement layers R3 and R4. Further, no failure surface which intersects with reinforcement layer R2 was evident. Hence at some point during 0.2g shaking step, the development of higher failure surfaces (reinforcement layers R4 and R3) was exchanged for the development of the lower failure surface (reinforcement layer R2). To inspect this, Section 5.5.3 plots the development of deformation within the 0.2g shaking step.

Figure 5-18 shows a vertical shearing surface, not readily visible in the vector plot of Figure
5-15. However, it is important to note that shear strain is calculated based on the differential displacement of patches. Hence the vertical shear surface is the result of differential displacement in the vertical components of the vector plot.

Figure 5-19 compares the failure surfaces determined using GeoPIV at 0.3g and 0.4g, with those visible in the images captured and used for GeoPIV. Obviously they compare well with that visible to the eye. The angle of the lowest failure surface is calculated from Figure 5-19 as 66° to the horizontal, steeper than that shown in Figure 5-2 of 55°. However, it was noted with regard to Figure 5-2 that the failure surface is not exactly planar, and was steeper at lower elevations.

![Comparison of failure surfaces](image)

**Figure 5-19. Comparison of location of failure surfaces plotted and seen within the image after 0.3g (a) and 0.4g (b).**

Some general comments are made with regard to the shear surfaces shown within the GeoPIV window, not visible by eye using the sand markers.

- GeoPIV analysis showed deformation to have occurred at the reinforced and retained backfill interface by 0.1g shaking, and prior to these becoming obvious using the sand markers at 0.3g.

- The existence of a near-vertical shear surface that appears to form at the back of the reinforced soil block.

- GeoPIV analysis identified high shearing at the tips of the reinforcement.
• GeoPIV analysis identifies the existence of shear surfaces which appear to enter the back of the reinforced soil block horizontally, and at an angle shallower than that within the retained backfill.

A summary of the development of deformation as understood for all walls is made in Section 5.5.5.

5.5.2 Interpretation of localised shear strains

In order to interpret the localised shear strains, it is easier, at least initially, to determine what sort of block displacement results from the plotted shear strain. Shear strain is defined in Equation 5-2.

\[ \gamma = \frac{\Delta \delta}{t} \]  

(5-2)

Where \( \gamma \) is the shear strain, \( \Delta \delta \), the differential displacement, and \( t \) is the thickness over which the displacement occurs, i.e. the thickness of the shear band.

The shear strains identified using GeoPIV in Figure 5-17 (d) were saturated at 50% strain because of the scale used. However, the shear band thickness is increased because a larger thickness of soil has shear strains equal to or greater than 50%; thus the calculation of differential displacement by rearrangement of Equation 5-2 remains the same. Figure 5-19 (d) is replotted at a different scale in Figure 5-20 to show the effect of strain saturation on shear band thickness.
Figure 5-20. Residual shear strain accumulated by the completion of 0.4g shaking step for Test-6 plotted with the maximum strain scale set to: (a) 50% and (b) 80%.

As seen in Figure 5-20 (a), by the completion of 0.4g, the shear strain accumulated in the lowest failure surface was at least 50% along a shear band approximately 15 mm thick. This equates to a block differential displacement of approximately 7.5 mm. In Figure 5-20 (b) the shear band where 80% shear strain is recorded is seen to be a thinner 10 mm thick. Hence the block differential displacement is approximately 8 mm. Note that the difference in block differential displacement is due to inaccuracy in determining a representative shear band thickness across the surface visible.

It is important to compare the displacement predicted by the GeoPIV strain plots with that observed visually, to determine the accuracy of the GeoPIV strain analysis. Hence the displacement along the lowest shear surface was measured from a photo taken similar to Figure 5-19 (b). In the image, the shear band creates a “kink” in the coloured horizontal sand line. Whilst it is difficult to measure the “kink” accurately due to the variable thickness of the horizontal sand layer, it still provides a good indication of the relative soil displacement. The slip movement was measured as 11 mm and agrees roughly with the more accurately determined 9.2 mm shown in the quiver plots presented in Section 5.5.1.

If the reinforced soil block behaved completely rigidly, then the horizontal component of the active wedge movement would be entirely transmitted to the wall face, and would be
recorded with the displacement transducers. The horizontal component of the slip movement is determined by trigonometry; the failure surface is inclined at 66° to the horizontal and at this angle, the lateral displacement of the active wedge is approximately 3 mm. This is significantly less than the 14.5 mm recorded at the wall face at 300 mm elevation as shown in Section 4.4.1. The discrepancy in lateral displacement recorded at the interface between the reinforced and retained backfill and at the wall face is due a number of reasons:

- Multiple sliding surfaces developed within the interface region, and the above calculations considered displacement along only one of these. In reality, displacement along all sliding surfaces contributes to displacement at the wall face.

- The reinforced soil block is not perfectly rigid, hence displacement of the active wedge is not propagated perfectly to the wall face.

- Displacement recorded at the wall face includes displacement of the active wedge and deformation incurred within the reinforced soil block.

- The local shear strains plotted in Figure 5-20 are shown only along a small region of the failure surface. The above calculations are based on average approximations of shear band thickness, inclination angle and shear strain.

5.5.3 Development of deformation within the 0.2g shaking step

GeoPIV analysis enabled shear strain to be recorded during the 0.1g and 0.2g shaking step of Test-6. During 0.1g, an inclined failure surface was seen to intersect with the end of reinforcement R3. However at some time during 0.2g, the development of this higher surface was surpassed by another inclined failure surface seen to extend down to the second layer of reinforcement, R2. Figure 5-21 shows the development of shear strain during the 0.2g shaking step of Test-6.
Figure 5-21. Shear strain accumulated during 0.2g shaking step and cycle peak: a) 1, b) 5, c) 10, d) 20, e) 30, f) 50. The cycle peaks are shown in the shake-table time-history (g).
Figure 5-21 shows the development of deformation *during* the 0.2g shaking step. There are several initial points to note with the derivation of the plots. Firstly, the shear strain is accumulated only during 0.2g shaking step, i.e. does not include any shear strain developed during the 0.1g shaking step. Secondly, the shear strain scale has been selected to coincide roughly with the likely shear strain developed at peak strength of Albany sand determined from previous triaxial tests conducted on the same Albany sand by Roper (2006). Hence, recorded shear strain larger than 8% indicate that the peak strength of the sand has likely been reached, and a degraded strength is likely to be mobilised along the shear surface. The shear surface could now be termed a ‘failure surface’.

The GeoPIV analysis based on residual strains accumulated by the end of each shaking step shown in Figure 5-17 (b) noted two inclined shear surfaces which extended from the wall crest down to the back of the reinforced soil block and reinforcement layers, R3 and R2 at the end of 0.2g shaking. The figure appeared to show that the lower shear surface developed first as it had the larger shear strain value.

However the development of deformation during 0.2g step is clearly illustrated in Figure 5-21 (a) – (d) which shows, contrary to the residual analysis, that the shear surface which intersects with reinforcement layer R3 is developed first, not the lower surface intersecting with reinforcement R2. The development of the higher shear surface is surpassed by the lower surface by the 20th cycle of shaking at 0.2g. Only the lower failure surface was observed by the sand markers on completion of the 0.3g shaking step.

The following points are made with respect to the development of the lowest failure surface during 0.2g shaking step as shown in Figure 5-21 above.

- By the end of the 5th cycle (b), a very thin band with 2 – 3% shear strain accumulated was visible at the end of reinforcement, R2.

- By the end of the 10th cycle (c), this shear band grew in width and length and propagated both upwards into the retained fill and horizontally along the reinforcement layer, R2.

- By the end of the 20th cycle (d), around 8% shear strain was recorded along the failure surface and propagation of shear strain continued as described above for the rest of the shaking step, and, increased shaking steps.
Additionally, Figure 5-21 shows a near-vertical shear surface tracing the back of the reinforced soil block by the 10th cycle with roughly 2% shear strain accumulated. It can be seen that the rate of shear strain development along the vertical surface increased once the lowest inclined shear surface likely surpassed peak strength.

5.5.4 Cyclic development of deformation

GeoPITV is used in this section to show the development of shear strain during the first complete cycle of 0.2g shaking step of Test-6. This is shown in Figure 5-22 below. Five images were selected according to the shake-table displacement time history shown in the figure, such that the shake-table displacement started at its initial positive (left) position. Note that the colour scale has been reduced to 2% of shear strain to better aid in its visualisation.
Figure 5-22. Shear strain developed within the first complete shake-table displacement cycle of the 0.2g shaking step for Test-6. The initial position of reinforcement layers R2, R3 and R4 are indicated with a dashed white line in (a).

It can be seen in Figure 5-22 (a) – (c) that, as expected, shear strain is developed within the interface region when the shake-table moves from left-to-right (positive to negative on the shake-table displacement time history). This corresponds to an acceleration time-history exactly opposite (from negative to positive). Hence outwards (positive) inertial forces are induced on the wall face as shown schematically in Figure 5-23.
Figure 5-23. Schematic showing the direction of shake-table displacement and the corresponding inertial force, $F_{IR}$, acting on the wall face.

Figure 5-22 (c) shows a peak strain of around 2% along the inclined shear surface. The wall is then displaced from the negative to positive (right-to-left) position, and this induces inertial forces acting in the opposite direction towards the backfill. These inertia forces act to stabilise the wall. Figure 5-22 (e) shows that some shear strain is recovered, in that the peak strain recorded along the shear surface is less than the previously recorded 2%.

In order to check whether the above trend of strain continues with further cycles, the strains accumulated within the first four cycles were plotted with GeoPIV and shown in Figure 5-25. Figure 5-24 shows the Displacement transducer (Disp 1) time history recorded at the wall face, and the timing of the images selected for the analysis. Peaks and troughs for cycles 1, 2, and 4 were selected because the facing displacement increased incrementally, whereas cycle 3 showed a reduced response which could have confused the analysis.

Figure 5-24. Selected peaks and troughs of facing displacement transducer Disp 1, for use in the GeoPIV analysis.
Figure 5-25. Shear strain developed within the complete first four cycles of the 0.2g shaking step for Test-6. The timing of the images was made to coincide with the peaks (a – c) and troughs (d – f) for cycles 1, 2, and 4, respectively, as shown in the attached facing displacement time history.

Figure 5-25 (a – c) show the strain accumulated by the facing displacement peaks, and (d – f) shows the strain accumulated by the facing displacement troughs. As seen, there was a steady accumulation of strain with each peak of cycles 1, 2, and 4 (a – c), and that some strain was recovered with each trough of the same cycle (a – d, b – e, c – f). Again, around 2% strain was localised at the reinforcement layer tips R3 and R4, and that the beginnings of horizontal and inclined failure surfaces became clear (c).
However, it should be noted that the plots show some scatter and the accumulation of strain at other locations is not totally clear. This is a function of the analysis accuracy (and low 2% scale selected), and possibly that the soil response itself was not initially well defined during the first few cycles of shaking.

### 5.5.5 Residual analysis on Tests-5 and 7 interface region

The residual shear strain accumulated at the completion of each shaking step for Test-5 and Test-7 are shown in Figure 5-27 and Figure 5-28 respectively. The “window” locations used by Cameras C1 and C2 for imaging are shown in Figure 5-26.

![Image](image_url)

**Figure 5-26.** The "windows" (identified by dashed white lines) used for image capture by cameras C1 and C2 in Test-5 (a) and Test-7 (b), used for subsequent analysis using GeoPIV.
Figure 5-27. Maximum shear strain in Test-5 reinforced at L/H = 0.9 accumulated by the completion of:
a) 0.1g, b) 0.2g, c) 0.3g, d) 0.4g, e) 0.5g and f) 0.6g shaking steps.
Figure 5-28. Maximum shear strain in Test-7 reinforced at L/H = 0.75 and inclined at 70° accumulated by the completion of: a) 0.1g, b) 0.2g, c) 0.3g, d) 0.4g, e) 0.5g and f) 0.6g shaking steps.

Because GeoPIV can enable shear strains to be identified within the soil at far smaller levels than that rendered visible by the black sand markers, the spatial and temporal development of deformation as well as the complete deformation picture could be visualised.

The general progression of deformation for the vertically faced models is summarised below:

- Shear surfaces first occur at the soil surface and propagate deeper into the deposit, while new shear surfaces continue to develop.

- An inclined shear surface propagates downwards from the wall crest towards the back of the reinforced soil block (Figure 5-27 (b)).

- The failure surface propagates vertically upwards along the back of the reinforced soil block (Figure 5-27 (c)).
• In addition to propagating upwards, the failure surface also propagates: a) along the reinforcement layer and/or, b) at a shallow angle into the reinforced soil block (Figure 5-27 (c)).

• At continued shaking, the failure surface undergoes increasing shear strain (Figure 5-27 (c) – (f)); or,

• At larger acceleration shaking, the process is repeated and deeper failure surfaces propagate downwards towards the back of the reinforced soil block. (The angle of inclination remains approximately the same.) (Figure 5-27 (c) – (f)).

Similarly the development of deformation for the inclined wall in Test-7 is depicted in Figure 5-28 and demonstrates that the behaviour follows a similar pattern. However, there seems to be a decreased vertical interaction between the reinforcement layers, possibly because the reinforcement ends are staggered due to a constant reinforcement length attached to the inclined wall face (Figure 5-28 (f)). It is possible that this leads to the increase in stability noted in Section 4. This is likely due to increased difficulty in forming an active wedge behind the reinforced soil block.

5.5.6 Comparison of shear strain development within the interface region

Figure 5-29 compares the development of strain accumulated by 0.3g shaking test for all Tests-6, 5 and 7. By the completion of the 0.3g shaking step, Test-6 (vertical wall reinforced L/H = 0.6) had one well defined failure surface with a maximum shear strain around 50% which intersects with reinforcement layer R2. In comparison, Test-5 (vertical wall, reinforced L/H = 0.9) had one defined failure surface at strain levels of around 20%, with the upper portion at 50% shear strain, and that this intersected with reinforcement layer R3. On the other hand, the inclined wall had one defined failure surface, at around 25% maximum shear strain, and this intersected with reinforcement layer R4.
Figure 5-29. Comparison of residual maximum shear strain after 0.3g shaking step for Test-6 (a), Test-5 (b), and Test-7 (c).

The inclined wall accumulated the lowest maximum shear strain by 0.3g when compared to the other vertical walls, and this trend continued throughout further shaking. For instance, at 0.6g prior to failure, only the top most failure surface within the inclined wall had maximum shear strain along its length of around 50%, as compared to the vertical wall Test-5, which shows all visible failure surfaces to exhibit maximum shear strains larger than 50%.

The comparison shows that the development of deformation was more limited in the case of the inclined wall, at both low and high levels of shaking. Further, it is important to note that the inclined wall was only reinforced at L/H = 0.75 (Test-7), shorter than that used for the vertical wall (Test-5 reinforced at L/H = 0.9). This demonstrates the significant improvement in performance able to be gained from wall inclination.
5.6 Deformation of the reinforced soil block

5.6.1 Shear surfaces within the reinforced soil block

Figure 5-30 shows the reinforced soil block regions for Tests-5, 6, and 7. The originally vertical coloured sand marker columns, all show some rotation, and this corresponds to simple shear deformation of the reinforced soil block.

Figure 5-30. Comparison of reinforced block regions post-failure illustrating strain localisations within the reinforced soil block: (a) Test-5, L/H = 0.9; (b) Test-6, L/H = 0.6; (c) Test-7, L/H = 0.75 and inclined 70° to the horizontal.
As observed by Watanabe et al. (2003) and Sabermahani et al. (2009) while the mode of failure is termed overturning, it actually comprises simple shear within each layer of the reinforced soil block, as the facing rotates about the toe. The mechanism of internal deformation of the reinforced soil block is shown in Figure 5-31.

![Figure 5-31. Simple shear deformation of the reinforced soil block.](image)

Additionally, Figure 5-30 also shows irregular discontinuities evident in the vertical sand columns for all tests. The FHR facing and overturning mode causes localised shear deformation across the thickness of each reinforcement layer. It is possible that this shear strain will accumulate at localised weaknesses (shear bands), and not necessarily the soil-reinforcement boundary. For instance, localised failure surfaces are evident both at the front (e.g. (b) within the circles) and back (e.g. (a - c) within the circles) of the reinforced soil block. Additionally, dislocations along the reinforcement are readily visible for the shorter, \( L/H = 0.6 \) reinforced soil block (b), but are not so apparent for the tests reinforced at a longer \( L/H = 0.9 \) and 0.75, (a) and (c) respectively.

Further, it can be seen for all tests that the ends of the reinforced block are dragged downwards as the wedge formed within the backfill slid down and along the failure surface. This was particularly visible for the longer reinforced Test-5 (Figure 5-30 (a)).

It should be noted that for the purposes of external stability and wave propagation, all design codes currently assume rigid block behaviour. This assumption is entirely contrary the behaviour observed above.

### 5.6.2 GeoPIV analysis of reinforced soil block

Figure 5-30 (a - c) shows that both simple shear, and localised strain deformation occurs along horizontal planes of the reinforced block for Tests-5, 6 and 7. To further investigate these observations, a second “window” wholly within the reinforced zone that straddled reinforcement layer R4, was investigated, as shown in Figure 5-9 and Figure 5-26. The 4th
layer of reinforcement was selected because design codes specify 60% of the wall height as the location of the largest dynamic earth pressure component.

For the GeoPIV analysis, patches sized 64 by 64 were used, larger than the 32 by 32 sized patches used for the analysis of the interface region above. While both cameras had similar resolution per square mm, the analysis with patches sized 32 by 32 for the reinforced soil region showed considerably more wild vectors than for the interface region. It is likely that the patches deformed during shaking, leading to erroneous displacement data. Additionally, some scratches were visible on the inside of the acrylic window, and this caused some patches to become ‘stuck’, rather than trace the actual soil displacement. Hence in order to overcome these limitations, patches sized 64 by 64 pixels were used for the analysis.

Figure 5-32 (a – d) shows the development of maximum shear strain at the completion of shaking steps 0.1g – 0.4g for Test-6. Appendix D displays this information at a lower scale.
Figure 5-32. Maximum shear strain within the reinforced soil block at reinforcement layer, R4, for Test-6 reinforced at L/H = 0.6 at the completion of: a) 0.1g, b) 0.2g, c) 03.g, d) 0.4g.

Figure 5-32 (a) shows that some small maximum shear strain of up to 2% developed in two regions located both near the front and back of the reinforced soil block (225 mm and 325 mm from the initial braced position of the wall face) within the first 0.1g shaking step. After 0.2g shaking (Figure 5-32 (b)), the maximum shear strain within these regions was increased to 4 - 6%, whilst much of the upper window was subjected to shearing of approximately 2%. Areas of previous shearing were further developed during 0.3g and 0.4g shaking up to around 10% shear strain, with a background (global) average maximum shear strain of 3 – 4%.

Figure 5-33 shows the high-speed camera images used for the GeoPIV analysis taken at the beginning of testing (a), and end of 0.4g (b) and 0.5g (c) shaking steps. Figure 5-33 (c) shows how the block translates almost more than 100 mm during the 0.5g shaking step, and hence why shear strain was plotted only up to 0.4g (for reasons as discussed in Section 5.5).
Figure 5-33. High-speed camera images used in the GeoPIV analysis of the reinforced soil block of Test 6: (a) before testing, (b) after 0.4g shaking step, and (c) after failure at 0.5g shaking. The vertical sand marker lines have been numbered to indicate the movement of these lines between shaking steps. Note that a strain localisation is visible in the 5th vertical marker line and has been circled.

Also shown in Figure 5-33 (c) is an apparent strain localisation (circled) visible in the coloured vertical marker line after the 0.5g shaking step. The GeoPIV analysis shown in Figure 5-32 shows a region of high shear strain after the 0.3g shaking step (c), near the back of the reinforced soil block from about 375 mm to 400 mm at an elevation of 550 mm to 590 mm. However, because the reinforced soil block translates and overturns significantly during the 0.5g shaking step this deformation has not been able to be shown clearly in the GeoPIV plots up to 0.4g.

Interestingly, a shear surface of similar elevation was also visible in the GeoPIV analysis of the interface region, post 0.3g, shown in Figure 5-17 (c). In this figure, an inclined surface appeared to enter the reinforced soil block at an elevation of approximately 580 mm. It is possible that this shear surface entered into the reinforced soil block and is shown in Figure 5-33 (c) though this is not clear.
In general, the maximum shear strain accumulated by 0.4g and plotted in Figure 5-32 (d) could be associated with the reinforced soil block undergoing simple shearing along horizontal planes (Watanabe et al. 2003). However, Figure 5-32 (a – d) indicates that the shear strain associated with this mechanism accumulated non-uniformly within the layer.

The simple shear within the reinforced soil block can be quantified using the rotational component of displacement recorded at the wall face. For instance, the rotational component of Test-6 calculated up to the 0.4g shaking step was 35 mm. Hence the average shear strain accumulated within the entire reinforced soil block of height 900 mm is calculated in Equation 5-3:

$$\gamma = \frac{\Delta \delta}{t} = \frac{35}{900} = 3.8\%$$  \hspace{1cm} (5-3)

The 3.8% shear strain is a theoretical value of shear strain accumulated within the reinforced soil block over the entire wall height. Similar maximum shear strains are shown in the GeoPIV analysis shown in Figure 5-32 (d), with some regions of maximum shear strain higher than 3.8% apparent, as well as regions where no shearing has occurred.

To better understand the behaviour of the reinforced soil block prior to failure, the maximum shear strain, \(\gamma\) can be divided into its components: shear strain, \(\gamma_{xy}\), and linear strains in the x and y directions, \(\varepsilon_x\) and \(\varepsilon_y\), respectively. Figure 5-34 plots the maximum shear strain, (replotted for comparison, a) and those strain components just noted (b – d).
Figure 5-34. Components of strain accumulated up to 0.4g shaking step for Test-6: a) Maximum shear strain, $\gamma$ (replotted for comparison), b) shear strain, $\gamma_{xy}$, c) linear strains in the horizontal direction, $\varepsilon_x$, and d) linear strain in the vertical direction, $\varepsilon_y$.

It can be seen in Figure 5-34 that the maximum shear strain (a) is comprised of shear and linear strains (b – d) combined by Equation 5-1. Figure 5-34 (b) shows a background shear
strain of 3 – 4%, which is consistent with that predicted assuming an average strain across the height of the wall. The linear strains in the horizontal direction are largely negative, which indicates extension of the soil along the layer of reinforcement as the block rotates out. The vertical linear strains are all positive, indicating compression in the vertical direction, and hence settlement of the soil within the reinforced soil block.

In order to understand the deformation during failure at 0.5g, GeoPIV was used to analyse the maximum shear strain accumulated within 0.5g shaking step. Figure 5-35 (a – e) plots the development of shear strain accumulated from the beginning of 0.5g, every 10th cycle through to the 50th cycle at the end of the test.
Figure 5-35 demonstrates that, during the final shaking step at 0.5g, there are three mechanisms of failure within the reinforced soil block. The first is that of whole scale sliding, and (a – e) shows the strain plot to translate from the back of the “window” at around 400 mm, to 315 mm away from the original wall face. The second mechanism is the accumulation of simple shear with the general rotation of the wall about the facing toe. For instance, within the first 30 cycles, up 10% maximum shear strain has accumulated in the top half of the window. The third mechanism, which appears dominant between the 30th and 50th cycles shows a horizontal high strain localisation along the reinforcement layer of up to 30%, while the rest of the window exhibits lower maximum shear strains of between 5% and 20%. This mechanism is consistent with “pullout failure” of the reinforcement from the surrounding soil.

The local response around reinforcement layer R4 exhibited in the GeoPIV plots above can be compared with the global response of the entire wall. Figure 5-36 shows the geometry of failure of the Test-6 wall face during the final shaking step.
Figure 5-36. Geometry and mode of failure of Test-6 during 0.5g (final) shaking step.

As can be seen in Figure 5-36, both sliding and rotation occur within the first 30 cycles of shaking. However, during the final 20 cycles, only rotation of the wall face is present. This is reflected in the displacement time history (Section 4.3.5) which shows the rate of facing displacement increase within the last 20 cycles, most likely caused by the facing weight, and the added soil resting on the wall, contributing to an increased overturning movement and hence rate of rotation. This could have also led to a decrease in sliding, as evident. The influence of the facing weight could have contributed to the pullout failure made visible using GeoPIV.

Test-5 reinforced the longest at L/H = 0.9, displayed similar behaviour as to Test-6, in that the deformation accumulated somewhat non-uniformly within the reinforced soil block due to block rotation. However, during the final shaking step at 0.7g, there was no evidence for pullout failure as shown in Figure 5-35. The GeoPIV analysis of the reinforced soil block for the Test-5 is presented in Appendix D.
5.6.3 GeoPIV analysis of inclined reinforced soil block

The Test-7 reinforced soil block, which was inclined at 70° to the horizontal demonstrated a higher component of sliding compared to the vertical wall tests. This could be expected to change deformation patterns within the reinforced soil block itself. Figure 5-37 plots the maximum shear strain calculated using GeoPIV centred on reinforcement layer R4 horizontally from 300 mm to 650 mm from the original position of the facing heel.
Figure 5-37. Maximum shear strain accumulated within the reinforced soil block by: a) 0.1g, b) 0.2g, c) 0.3g, d) 0.4g, and e) 0.5g shaking steps, centred on reinforcement layer, R4, for Test-7 reinforced at L/H = 0.75 and inclined at 70° top the horizontal. Note that the 0.7g shaking steps are not plotted due to the large sliding component during these steps, impacting on the results as discussed.

Figure 5-37 shows that from 0.1g to 0.6g shaking step (a – c) regions of maximum shear strain around 10% are concentrated at the front of the “window”. Because the wall is inclined, at 600 mm elevation, the front of the “window” is only 80 mm from the back of the wall face, and this could led to the strain localisations here. Contrary to Test-6, no strain localisation along the reinforcement layer R4 (indicated by the dashed line in (a)) is apparent. The development of maximum shear strain during the final 0.7g shaking did not display any pull-out failure, and is presented in Appendix D.

Instead, Figure 5-37 (a – f) shows by the completion of 0.6g, a background (global) shear
strain of at least 3 - 4%, with pockets of high localised shear strain likely due to settlement proximate to the wall face. The background simple shear is generated by the rotation of the wall and was determined in Section 4.5.2 as 23.9 mm, which, over the 900 mm height of the wall corresponds to 2.6% shear strain. This strain is far lower than that observed using GeoPIV.

5.7 Idealised model for progression of deformation

5.7.1 General model of deformation for vertical walls

Two methods have been presented to show deformation pre- and post-failure. GeoPIV has been used to quantify small strains in selected regions prior at low shaking, and the horizontal and vertical sand markers have been used to show global mechanisms of deformation during and after ultimate failure. The complementary methods have been used to build a complete picture of the mechanisms of deformation for the GRS models under investigation. Figure 5-38 presents a general schematic of the development of deformation within a GRS wall undergoing increasing base input acceleration.

![Diagram of deformation progression](image)

Figure 5-38. Idealised model of progression of deformation mechanism for GRS model walls undergoing increasing stages of seismic excitation from (a) to (e).
An explanation of the progression of deformation with increasing base excitation shown in Figure 5-38 is made with the following:

a) The GRS wall prior to seismic excitation.

b) Under the application of a small base acceleration, the wall rotates slightly, and concurrently a shear band begins to develop which extends from the wall crest down towards the tail of the top most layer of reinforcement, R5. A vertical shear band also starts to develop. Once the reinforcement tail has been reached, the shear surface is propagated horizontally along the weakest surface which could be either the reinforcement, or at some angle into the back of the reinforced soil zone, or both.

c) Application of a higher acceleration causes another shear surface (at the same angle as before) to extend from the wall crest further within the backfill, down towards the back of the reinforced soil block to intersect with the tail of reinforcement layer, R4. Lateral movement of the face allows the active wedge to slide downwards, and this generates settlement behind the reinforced soil block.

Other shear bands beginning at the wall crest and inclined downwards to the tail of the reinforcement layer R4 are developed as the wall overturns.

d) With application of continued and increasing base acceleration the wall overturns (with further settlement behind the reinforced soil block) and further shear surfaces are developed within the backfill. These are propagated downwards to intersect with the tails of reinforcement layers R3, R2 and R1.

Again, once the shear surface has reached the localised weakness of the reinforcement tail, the surface is propagated either horizontally along the reinforcement, at some angle into the back of the reinforced soil block, or both.

e) Upon application of the critical acceleration, the final and lowest shear surface is developed until a failure surface is formed. In this case, the rigid foundation confines the soil at this elevation, and the weakest location for the failure surface to propagate is along the reinforcement layer R1.

Continued, or shaking larger than the critical acceleration, causes the active wedge to slide along the lowest formed failure surface. The wall face can then overturn. The active
wedge then slides into the back of the reinforced soil block; generating the largest settlement just behind the reinforced soil block. This movement may also drag the reinforcement tails downwards.

Note that depending on the exact size of each base acceleration step, various shearing surfaces may or may not develop. For instance under a larger acceleration than that applied in (b), the upper shearing surfaces may be skipped, to develop ones at a lower elevation.

Two points concerning the failure surface angles, and the absence of sliding prior to failure requires some explanation:

Firstly, MO theory predicts that the angle of the failure surface is reduced (becomes shallower) with increasing seismic coefficient of acceleration. Such is the case for a conventional gravity retaining wall, where the failure surface originates from the toe of the wall. However, in the current model tests on GRS walls with FHR panel facing, excepting the inclined test (Test-7), the angle of all failure surfaces were reasonably parallel, depending on the L/H ratio. It may be possible that the failure surface angle is related to the critical acceleration of each model, which depends upon stability due to the L/H ratio. For instance, because Tests-6 and 5 were reinforced differently, they failed at different acceleration levels, and hence developed shearing surfaces at different angles.

In contrast, despite different reinforcement L/H and geometry, Tests-5 and 7 both failed at 0.7g, having the same critical acceleration value. The failure surface angles formed in the final shaking step during failure at 0.7g were similar (~ 40°). However, as shown in Figure 5-4, prior to 0.7g, Test-7 failure surfaces were shallower still at around 30°. The difference between the initial angles recorded for Test-7, and those formed subsequently, may be explained by considering that a structure’s critical acceleration changes during an earthquake, with the associated change in soil properties and geometry. As the wall became more vertical after increasing shaking, its critical acceleration changed to that of Test-5, hence similar failure surface angles were formed.

Secondly, in the above idealised model, no sliding of the base occurs until a failure surface had formed which intersected with the lowest layer of reinforcement, effectively allowing the reinforced soil block to slide. This was largely verified by the experiments which show only minimal sliding at the wall face until failure occurred. This has also been noted in shake-table experiments reported by Koseki et al. (2006).
Additionally, because the foundation was rigid, there was no subsoil deformation which could contribute to base sliding. Hence, any sliding at the wall face, prior to the formation of the failure surface intersecting with the lowest reinforcement layer could be ascribed to soil-reinforcement interaction. That is, some “pullout” of the reinforcement has to occur before there can be sliding at the toe, and this was not readily seen.

5.7.2 Comparison of idealised deformation model with other research

The general progression of deformation at low base accelerations (and therefore low strain levels) has been identified using GeoPIV. At these low strain levels, the shearing surface is often not visible with the use of coloured sand marker lines, and these are only capable of showing the development of deformation at larger strains, when “kinks” in the sand markers can be seen. Consequently, sand markers can readily identify progression of deformation near failure, and have been used in both the current and other experimental studies (Sabermahani et al. 2009; Watanabe et al. 2003) to identify failure mechanisms.

The general progression of deeper shear surfaces with increasing base acceleration has been observed in experimental studies by Watanabe et al. (2003) and Sabermahani et al. (2009). For instance, two out of the three tests conducted by Watanabe et al. (2003), display two failure surfaces, and in one test, the failure surfaces are not parallel. However, in numerous model tests (11 where the wall failed by overturning and not bulging, out of a total of 20 model tests) the multiple failure surfaces were reported to be parallel, largely confirming the above model.

Additionally, all of the failure surfaces observed by Watanabe et al. (2003) intersected with the lowest layer of reinforcement. In contrast, the failure surfaces observed by Sabermahani et al. (2009) did not extend to the bottom most layer of reinforcement. The difference in behaviour was most likely due to the difference in facing type. The former walls were faced by FHR panel, whilst the latter walls were faced by wrap-around facing. As stated in Section 1.4.2, Tatsuoka (2008) listed one of the main advantages of the FHR panel facing being that all reinforcement layers act in unison to resist deformation. Hence, it is likely that the wrap-around facing models (Sabermahani et al. 2009) failed before the bottom most layer of reinforcement was engaged, and thus no failure surface propagated down towards the bottom most layer of reinforcement.
5.7.3 General model of deformation for inclined walls

The inclined wall follows largely the model as proposed for the vertical wall, except that, as noted in Section 5.6.3, there is reduced interaction between the horizontal layers of reinforcement, and no active failure wedge could form readily. It is for this reason, that the inclined wall demonstrated an increased stability, and a reduced displacement response of the wall top when compared to the other, vertical walls (where a near-vertical shear surface, and hence active wedge, could form easily).

In practice, the top few layers of reinforcement for GRS walls (vertical or otherwise) are often extended, to provide increased wall stiffness and prevent overturning failure (Tatsuoka 2008). In one of the model tests conducted by Watanabe et al. (2003), two of the reinforcement layers were extended; the top most and one just above mid-height of the model. This increased the acceleration required to generate failure by almost 0.1g more than the model with without any extended layers, but same other parameters. Watanabe et al. (2003) noted that the extended reinforcement resisted the formation of the failure plane, and governed strongly its location. It is likely that a similar mechanism contributed to the improved stability of the inclined model in the current tests, when compared to the vertical walls.

5.8 Summary

Deformation within the GRS wall models was observed using a combination of coloured sand markers and GeoPIV. The former was found to be effective at recording mechanisms of ultimate failure, which by definition, occur at medium to large strains. The latter was found able to illustrate deformation at much smaller strains than that visible by eye. The GeoPIV analysis was able to readily show considerable deformation which had occurred during small amplitude shaking, previously not visible using only coloured sand marker lines.

GeoPIV was used to plot the strain field of two regions: the interface between the reinforced soil zone and retained backfill zone. For the interface region between zones, GeoPIV was used to show the development of deformation for the entire test of increasing acceleration shaking steps. This identified the existence of shearing surfaces at higher elevations within the retained backfill and at low acceleration levels.

For all models, failure was predominantly by overturning, with some sliding component.
This was accompanied by the development of inclined shear surfaces which started at the wall crest and extended down to the back of the reinforced soil block. With increasing acceleration amplitude, further surfaces were developed progressively deeper within the retained backfill. Vertical surfaces were also propagated near-vertically up the back of the reinforced soil block to allow the creation of an active failure wedge, and either along the reinforcement layer, or at some angle into the reinforced soil block.

Sliding was very small for accelerations less than the critical acceleration level, whereas at accelerations larger than this, sliding became significant. Failure of the model was observed when a failure surface developed (i.e. with associated post-peak reduction in shear resistance) from the wall crest and was inclined downwards towards the back of the reinforced soil block and intersected with the lowest layer of reinforcement. This mechanism then allowed the model to slide forward, with further overturning.

Failure was accompanied by the active wedge sliding downwards and into the back of the reinforced soil block along the failure surface, and as a result, the highest settlement occurred just behind the reinforced soil block. The block also exerted down-drag forces on the tails of the reinforcement layers, and these were observed to have dipped downwards after testing.

Interestingly, a near-vertical shearing surface, visible in all of the vertical walls tested, was not visible in the GeoPIV analysis of the inclined wall (Test-7) for low accelerations. Because the inclined wall had the same length of reinforcement, the ends of reinforcement did not line up. It is likely that this made it difficult for a vertical shearing surface, and thus the active wedge to develop, and this would have contributed to the increased stability of the inclined wall compared to the vertical walls. It should be noted that a near-vertical surface behind the reinforced soil block became visible at failure, as the wall had rotated enough for such a surface to form more easily.

GeoPIV was also used to show the development of deformation during a shaking step, and within a single cycle. The latter showed that deformation of the interface region was related to the inertia force acting on the FHR aluminium panel facing.

The second region considered by GeoPIV was the reinforced soil block, and this was found to be non-rigid for all tests. Whilst the mode of failure is classified as overturning, simple shearing of the reinforced soil block was visible along horizontal planes, not necessarily along the reinforcement layers. For instance, small dislocations in the vertical colour sand
marker lines in between reinforcement layers were visible. GeoPIV showed that the shear strain occurred non-uniformly across reinforcement layer R4, and that pockets of shear strain larger than that predicted using plane strain theory was recorded. This has ramifications for design, where the reinforced soil block is considered a rigid composite mass.

Additionally, pullout failure was shown to occur for one wall reinforced the shortest, during ultimate failure. This was likely due to the geometry of failure and an increase in the overturning moment acting on the wall face due to the facing weight, and soil resting on the back of the facing. This demonstrated the importance of facing weight contribution at during overturning failure.

The failure mechanisms observed agree largely with the two-wedge model proposed by Horii et al. (1994), and the results observed by coloured sand marker lines and GeoPIV were combined to create an ideal model for the progression of deformation for GRS walls with a FHR facing.

References


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


CHAPTER 6
CONCLUSIONS AND RECOMMENDATIONS

6.1 Geosynthetic Reinforced Soil Walls

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

Experience in previous earthquakes such as Northridge (1994), Kobe (1995), and Ji-Ji (1999) indicate good performance of reinforced soil retaining walls under high seismic loads. During these earthquakes, significant damage of conventional retaining wall structures was reported, whilst reinforced soil structures demonstrated limited to no damage (Ling et al. 2001; Sandri 1997; Tatsuoka et al. 1996). However, this good performance is not necessarily due to advanced understanding of their behaviour, rather this highlights the inherent stability of reinforced soil against high seismic loads and conservatism in static design practices (White and Holtz 1997).

Hence the seismic performance of GRS walls has been the object of investigation in this study, and in particular the observation of the mechanisms and development of deformation
prior to failure. The influence of the L/H ratio and the inclination of the wall on seismic
performance was also examined as these are two important design parameters.

A series of seven reduced-scale model tests was conducted using the University of
Canterbury shake-table. The L/H ratio and wall inclination was varied from test to test and
wall facing displacement, acceleration within the backfill, and deformations of the wall were
measured during testing at varying levels of shaking intensity.

6.2 Conclusions from the experimental study

The specific objectives of the project were to:

- Develop testing procedures for shake-table tests on GRS walls.
- Quantify the influence of the L/H ratio and wall inclination on seismic behaviour.
- Identify deformation patterns and failure mechanisms of GRS walls under seismic
  loading.
- Identify critical issues for further research studies.

6.2.1 Shake-table tests

Based on previous model studies on reinforced soil, a strong box 3.0 mm long, 0.8 m wide
and 1.1 m high was constructed. The box included a 20 mm thick transparent acrylic
window, which enabled visual observation of failure patterns within the physical model. A
procedure for the construction and testing of reduced-scale GRS wall models on the
University of Canterbury shake-table was also developed.

The five most important considerations for the experimental design were as follows:

- The model was constructed in 12 layers, each 75 mm thick, up to the required 900 mm
  soil deposit height. The soil deposit (Albany sand) was compacted by vibration of each
  layer by the shake-table and a weighted steel plate placed on top of the layer. The wall
  face was braced during model construction, which was then removed prior to testing. A
  reasonably consistent model relative density was achieved across all tests, and the
  average relative density of the models was 92%.
• The model was faced by an FHR panel, and the model was reinforced with a stiff Microgrid reinforcement. A rigid connection between the facing and reinforcement was ensured. These details were representative of GRS walls with FHR facing for the observation of deformation patterns and ultimate failure.

• In addition to accelerometers within the backfill and displacement transducers at the wall face, the experiment also utilised high-speed cameras to record deformation within the backfill during testing. The captured images were analysed using GeoPIV for interpretation of deformation prior to, and during failure.

• Care was taken to develop procedures for experimental details such as model preparation, the seal between the wall face and box sidewalls, and construction of vertical and horizontal coloured sand marker lines.

• A series of seven model tests were conducted and the reinforcement ratio $L/H$ and the wall inclination were varied. Facing displacements, accelerations within the backfill, settlements, as well as deformation observations within the backfill were made. Importantly, the experimental design allowed the effect of model parameters such as $L/H$ and wall inclination on model behaviour to be observed.

### 6.2.2 Influence of $L/H$ ratio and wall inclination on seismic performance

The experimental series involved a parametric study which varied the $L/H$ ratio between $L/H = 0.6, 0.75$ and $0.9$, and wall inclination from the vertical, to 70° with the horizontal over four tests, namely Tests-1, 5, 6, and 7. The testing regime involved subjecting the models to successively increasing base excitation in 10 second duration steps, with a sinusoidal acceleration wave of frequency 5 Hz. At each stage, the amplitude of shaking was increased by 0.1g, with the initial shaking at 0.1g. The following conclusions are made with respect to an increase in the $L/H$ ratio from $L/H = 0.6$ to 0.9, across Tests-6, 1, and 7, respectively.

• An increase in the $L/H$ ratio increased both the critical acceleration (and by definition, the acceleration at which ultimate failure occurred) from 0.4 to 0.6g. Thus a 50% increase in $L/H$ ratio resulted in an increase of the acceleration level at which failure occurred by 40%.
• An increase in the L/H ratio caused the displacement-acceleration curve to be shallower, and hence the wall deformed less at low levels of acceleration.

• An increase in L/H ratio resulted in a reduction in the amplification of accelerations within the backfill prior to, and during failure. This can be attributed to the longer reinforcement generating an increase in stability.

The following conclusions are made with respect to a decrease in wall inclination from the vertical (Tests-1 and 7).

• A decrease in wall inclination from 90\(^\circ\) to 70\(^\circ\) to the horizontal, increased the critical acceleration (and by definition, the acceleration at which ultimate failure occurred), from 0.5g to 0.6g.

• A decrease in wall inclination caused the displacement-acceleration curve to be shallower, and hence to deform less at low accelerations levels, than the vertical walls.

6.2.3 Development of deformation

All models were observed through the transparent acrylic window on one side of the strong box during testing. For all models, failure was predominantly by overturning, with some sliding component. For all tests, sliding was very small for accelerations less than the critical acceleration level. For accelerations larger than the critical acceleration level, sliding was significant.

Two methods for the observation of deformation were employed. The first of these methods utilised horizontal and vertical coloured marker lines of sand placed within the backfill during construction. Dislocations in the coloured marker lines were able to show failure mechanisms which formed at medium to large strains.

The second of these methods utilised high-speed camera imaging during testing of two regions within the soil deposit: 1) centred on the second reinforcement layer from the top, and 2) the interface between the reinforced soil block and the retained backfill. These images were then analysed with Geotechnical Particle Imaging Velocimetry (GeoPIV) software. GeoPIV was able to show deformation which developed at low acceleration levels (small
strains), not visible by the naked eye. GeoPIV therefore provided a more complete picture as to the development of deformation.

With both of these methods, an idealised model for the progression of deformation with increasing base acceleration was presented. The model basically confirms the use of the two-wedge mechanism for ultimate failure.

Overturning of the wall was observed to be accompanied by inclined shear surfaces which started at the wall crest and extended down to the back of the reinforced soil block. With increasing acceleration amplitude, further surfaces were developed progressively deeper within the retained backfill. Upon reaching the back of the reinforced soil block, these surfaces were propagated upwards to allow the creation of an active failure wedge, and either along the reinforcement layer, or at some angle into the reinforced soil block.

Failure of the model was only initiated when a failure surface developed (i.e. with associated post-peak reduction in shear resistance) from the wall crest and was inclined downwards towards the back of the reinforced soil block and intersected with the lowest layer of reinforcement. This mechanism then allowed the model to slide forward, with further overturning, until failure occurred. This was accompanied by the active wedge sliding downwards and into the back of the reinforced soil block along the failure surface, and as a result, the highest settlement occurred just behind the reinforced soil block.

The inclined model demonstrated similar behaviour, with only slightly more sliding components before and after failure. For instance, sliding contributed to 40% of the total displacement of the wall top, compared to only 30% for the vertical walls. Again, the active wedge slid down the failure surface and into the back of the reinforced soil block, however this movement was observed to be slightly rotational.

With the inclined test, the length of reinforcement was kept constant, and this resulted in a back-tilted reinforced soil block, with the ends of reinforcement also inclined. It should be noted that vertical failure surfaces appeared to form with difficulty because of this. Hence an active failure wedge was not formed easily until the wall had overturned some, and this may have contributed to it having the shallowest and lowest acceleration-displacement curve, and resultant increased stability.
The reinforced soil block was found to be non-rigid for all tests. Whilst the mode of failure is classified as overturning, a combination of simple shearing of the entire reinforced soil block, and localised shearing along horizontal planes was visible. This shearing deformation was found to occur, at least at high elevations (the reinforcement layer second from the top) non-uniformly within each layer. For instance pockets of shear strain larger than that predicted using an average shear strain up the wall were developed, and horizontal dislocations were visible along the reinforcement layer or between reinforcement layers near the back of the reinforced soil block. This was combined with the tail of the reinforcement being dragged downwards as the active wedge slid down into the back of the reinforced soil block.

6.2.4 Implications to design

Deformation prior to ultimate failure

A major assumption for design purposes is that the reinforced soil zone behaves as a rigid composite block. However, experimental observations show that overturning failure is accompanied by inclined shearing surfaces within the retained backfill, and along horizontal planes within the reinforced soil block. Further, this deformation occurs even at low acceleration levels. Deformation of the reinforced soil block was shown to be non-uniform between each layer of reinforcement, and not necessarily along the reinforcement layers.

Wall inclination

A reduced wall inclination was found to contribute to the stability of the wall significantly. Additionally, even after multiple shaking steps, deformation was small and the wall was likely to be serviceable. This is because pre-failure, the wall deformed by overturning, and this only increased the inclination of the wall towards the vertical. Therefore the wall did not actually overturn past the vertical before (and during) failure.

Acceleration Amplification

Amplification of acceleration was found for all tests to be non-linear up the wall face, and to generally increase with increasing base input acceleration. An increase in the L/H ratio was effective at reducing this amplification.
The New Zealand Guidelines \textit{(Murahsev, 2003)} specify a maximum amplification factor of 1.3, and a downwards trend of amplification with increasing base acceleration. An opposite trend of increasing amplification with increasing base acceleration was observed in the current experiments with an FHR panel facing.

6.3 Model limitations

As with all model tests, there are limitations as to the applicability of results generated at model scale to full-scale. An attempt at model similitude was made to generate behaviour representative of prototype scale. Of particular concern was the scaling of soil properties associated with model studies at 1-g, and this could impact the failure mechanisms observed. However as the model was only able to fail by overturning and sliding (and pullout failure could not be quantified) it is unlikely that this had a major effect on the mechanisms observed.

Further, possible modes of failure were constricted to those three stated, and where other modes are allowed (by careful scaling of the soil-reinforcement interaction, inclusion of foundation sub-soil, etc) this could impact on the failure patterns observed.

It should be noted that the progression of deformation observed is only valid for GRS models with an FHR facing panel, as the facing rigidity ensured that all reinforcement layers were engaged in actively resisting deformation of the wall. Other facing types would result in different failure patterns to be observed.

Additionally, the model was subject to boundary effects, though where possible, these were minimised. For instance, acceleration data was recorded at the box centreline. However, it should be noted that deformation observations made at the sidewall of the box could have incurred the maximum possible boundary effects. It should be repeated that Watanabe et al. (2003) showed that there was a difference in angle of the failure planes measured at the transparent sidewall and along the centreline of the box, but not in the timing of formation. However it is unlikely that such a shift in angles would impact on the actual mechanisms observed.
6.4 Recommendations for future research

This was the first study of a longer research programme which investigates the seismic performance of GRS walls. Hence, much work was devoted to the experimental detail and methods of model preparation to enable future studies to possibly investigate, among other things:

- The use of local soils as backfill, and its impact on seismic performance.
- The addition of a surcharge load and its effect on deformation during seismic excitation.

The high-speed camera imaging employed in this study generated considerable data for subsequent analysis using GeoPIV. Additionally, the data can be presented in a variety of ways depending on the information required. For instance, total maximum shear strain, volumetric strain, linear strains and shear strain can be isolated and scrutinised. However, due to time constraints, not all of the data has been analysed in such a comprehensive manner, and only the most interesting results have been presented in this study. Hence, with regards to the GeoPIV analysis, the following points are noted:

- The strain data obtained by GeoPIV for both “windows” and all tests could be further scrutinised using the abovementioned components of strain to better isolate the mechanisms of deformation.

- The deformation of the reinforced soil block was not able to be quantified exactly using GeoPIV for all tests due to camera resolution and the location chosen for investigation (at 66% of model height). However, the use of GeoPIV was able to identify deformation within the reinforced soil block-retained fill interface well, and it is likely that a redesign of the camera location and/or resolution used for investigation of the reinforced soil block could be met with better results.

- Failure was governed by the development of a failure surface which intersected with the lowest layer of reinforcement. However, the reinforced soil block – retained backfill “window” was located too high to visualise the development of this deformation. The “window” could be lowered in future tests to rectify this.
• Camera resolution is governed by the camera’s RAM memory, and hence the selected frame rate. The analyses in this study showed that much information can be obtained from residual analysis of images taken just before and after shaking, i.e. without the need for high-speed imaging. Thus it would be possible to capture larger images at a higher resolution separately before and after testing, for analysis of residual deformation using GeoPIV. Subsequent imaging at 200 fps (and reduced resolution and window size) could then continue as per normal, to analyse response during shaking.

Recommended research to further investigate the influence of L/H ratio and wall inclination on seismic performance includes:

• The existence of an ‘optimum’ L/H ratio, whereby an increase in reinforcement length, is not met by a proportional increase in seismic performance.

• The effect of wall inclination on behaviour could be better investigated with further tests conducted on inclined walls with a more gradual transition of 10° increments from the vertical case.

• The Microgrid reinforcement used in the current tests was ‘inextensible’ at model scale. No comparison was made with other reinforcement types, and hence it is difficult to ascertain the influence of this parameter on seismic performance. This could be investigated with further testing using a larger range of reinforcement types and stiffness’s.

Finally, it is envisaged that the depiction of mechanisms of deformation can lead to more accurate performance based methods of design. For instance, while Newmark-type rigid-block methods have been shown to be reasonably accurate at predicting deformation during failure (Matsuo et al. 1998), the present study demonstrated that deformation occurred even at low levels of shaking intensity, predominantly by rotation of the wall about the toe. Given that these mechanisms of deformation at low levels of shaking have largely been identified in this work, future research can focus on the development of performance-based methods to quantify small-strain deformation, to aid in better prediction of seismic performance that meets SLS criteria.

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References


APPENDIX A

WALL DESIGN AND SOIL PROPERTIES
A.1 Model Wall Design

The model wall was designed at prototype scale, initially setting $L/H = 0.75$.

**Wall geometry at prototype scale**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall height</td>
<td>$H = 4.50,\text{m}$</td>
</tr>
<tr>
<td>$L/H$ ratio</td>
<td>$0.75$</td>
</tr>
<tr>
<td>Reinforcement length</td>
<td>$L = 3.375,\text{m}$</td>
</tr>
<tr>
<td>Inclination</td>
<td>$\theta = 90^\circ$</td>
</tr>
</tbody>
</table>

**Microgrid properties**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factored allowable tension</td>
<td>$T_{\text{fl}} = 14,\text{kN/m}$</td>
</tr>
</tbody>
</table>

**Soil Properties**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight of fill</td>
<td>$\gamma_f = 16.7,\text{kN/m}^3$</td>
</tr>
<tr>
<td>Internal friction angle</td>
<td>$\phi = 33^\circ$</td>
</tr>
<tr>
<td>Active Earth Pressure Coefficient</td>
<td>$K_{\text{ef}} = 0.295$</td>
</tr>
<tr>
<td>Angle of failure plane angle</td>
<td>$\psi = 61.5^\circ$</td>
</tr>
</tbody>
</table>

**Static horizontal wall thrust, $F_A$**

$$F_A = 0.5 \, K_{\text{ef}} \, \gamma_f \, H^2$$

$$F_A = (0.5 \times 0.295 \times 16.7 \times 4.5^2) = 74.7\,\text{kN/m}$$

**External stability: Check sliding failure**

$$F^*_h \leq \Phi \, F^*_v \tan \phi$$

$$F^*_h \leq (1.0 \times 253 \times \tan(33)) = 165\,\text{kN/m} \quad \text{SLIDING OK}$$

**External stability: Check overturning failure**

$$M^*_d \leq \Phi \, M^*_r$$

$$M^*_d = (74.7 \times 0.5 \times 4.5) \leq \Phi \, M^*_r = (0.75 \times 253 \times 0.5 \times 3.375)$$

$$M^*_d = 168\,\text{kN/m} \leq \Phi \, M^*_r = 321\,\text{kN/m} \quad \text{OVERTURNING OK}$$
Check number of reinforcement layers, \( N \), required

\[
N = \frac{F_A}{T_{ai}} = 5.3
\]

Hence 5 Layers of reinforcement, spaced vertically by 0.75 m has been selected.

**Internal stability: Check pullout failure**

Maximum tensile force in reinforcement, \( F^*_j \):

\[
F^*_j = \sigma^*_{bj} S_{vj}
\]

Factored design soil interaction strength against pullout, \( T^*_{di,j} \):

\[
T^*_{di,j} = \Phi \Phi_n F^* \alpha \sigma^*_{vi} L_e C
\]

Where, \( \Phi \) is the uncertainty factor for soil-reinforcement interaction (assumed 0.8), \( \Phi_n \) reduction factor associated with consequence of failure (1.0), \( F^* \) is pullout resistance (manufacturer tested as 0.8), \( \alpha \) is the scale correction factor (1.0), \( L_e \) is the reinforcement length in the resistant zone, and \( C \) is the reinforcement effective unit perimeter (2.0 for geogrid and geotextiles).

Hence, the reinforcement strength must satisfy:

\[
F^*_j \leq T^*_{di,j}
\]

<table>
<thead>
<tr>
<th>Reinforcement Layer</th>
<th>( L_a^1 (m) )</th>
<th>Vertical stress at layer, ( \sigma^*_{vi} (Kn/m^2) )</th>
<th>Factored soil interaction strength, ( T_{di}(Kn/m) )</th>
<th>Max Tensile force in reinforcement, ( F^*_j (Kn/m) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.97</td>
<td>62.6</td>
<td>268</td>
<td>13.9</td>
</tr>
<tr>
<td>2</td>
<td>2.56</td>
<td>50.1</td>
<td>185</td>
<td>11.1</td>
</tr>
<tr>
<td>3</td>
<td>2.15</td>
<td>37.6</td>
<td>117</td>
<td>8.31</td>
</tr>
<tr>
<td>4</td>
<td>1.75</td>
<td>25.0</td>
<td>63</td>
<td>5.54</td>
</tr>
<tr>
<td>5</td>
<td>1.34</td>
<td>12.5</td>
<td>24</td>
<td>2.77</td>
</tr>
</tbody>
</table>

\( ^1 \) Calculated from assumed failure surface angle, \( \psi = 61.5^\circ \)

PULLOUT OK
A.2 Seal Friction Measurements

Reduction of the seals influence on model behaviour involves minimising the friction force between model wall facing and box sidewall. The friction force is calculated as below:

\[ F_f = \mu_s F_N \]

Where \( F_f \) is the friction force, \( \mu_s \) is the static coefficient of friction, and \( F_N \) is the force normal to the box sidewall. The normal force against the box sidewall comprises the earth pressure force along the vertical length of the seal, in addition to the outwards force provided by the seal against the box sidewall. Thus reduction of the friction force can occur by reducing the coefficient of static friction and / or reduction of the normal force provided by the seal. While we cannot change the earth pressure on the seal, appropriate material selection, and a reduction of the seal's contact area will reduce the friction against the side wall.

**Static coefficient of friction**

The friction between various materials, Ployester plastic (PE), Teflon tape (TE), grease and steel was trialed via simple tilt table tests, and the static coefficient of friction, \( \mu_s \), calculated by:

\[ \mu_s = \tan \theta \]

The test results are shown in Table A-1.
Table A-1. Tilt-table tests on different seal materials to determine coefficient’s of friction

<table>
<thead>
<tr>
<th>Material</th>
<th>Trial 1</th>
<th>Trial 2</th>
<th>Trial 3</th>
<th>Trial 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>PE - PE</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\theta$</td>
<td>22</td>
<td>23</td>
<td>23.5</td>
<td>21</td>
</tr>
<tr>
<td>$\mu_s$</td>
<td>0.404</td>
<td>0.42</td>
<td>0.44</td>
<td>0.38</td>
</tr>
<tr>
<td>Teflon – PE (semi-rough) contact</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\theta$</td>
<td>20</td>
<td>13</td>
<td>18</td>
<td>18</td>
</tr>
<tr>
<td>$\mu_s$</td>
<td>0.36</td>
<td>0.32</td>
<td>0.32</td>
<td>0.32</td>
</tr>
<tr>
<td>Teflon – PE (smooth) contact</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\theta$</td>
<td>16</td>
<td>16</td>
<td>16</td>
<td>16</td>
</tr>
<tr>
<td>$\mu_s$</td>
<td>0.28</td>
<td>0.28</td>
<td>0.28</td>
<td>0.28</td>
</tr>
<tr>
<td>Teflon – greased PE (semi-rough) contact</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\theta$</td>
<td>23</td>
<td>23</td>
<td>22</td>
<td>21</td>
</tr>
<tr>
<td>$\mu_s$</td>
<td>0.42</td>
<td>0.42</td>
<td>0.404</td>
<td>0.38</td>
</tr>
<tr>
<td>Teflon – steel contact</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\theta$</td>
<td>18</td>
<td>18</td>
<td>18</td>
<td>18</td>
</tr>
<tr>
<td>$\mu_s$</td>
<td>0.32</td>
<td>0.32</td>
<td>0.32</td>
<td>0.32</td>
</tr>
</tbody>
</table>

Hence the Teflon PE (smooth contact was selected) as explained in Section 3.3.3.

A spring force was attached to the top of the facing panel whilst inside the box. Multiple spring force trials were made on the seal comprising a Teflon and PE contact, and only 1 N of force was required to generate wall rotation about the toe.
A.3 Particle Size Distribution

![Graph showing Particle Size Distribution of Albany Sand](image)

**Figure A-1. Particle Size Distribution of Albany Sand**

Coefficient of Uniformity $C_U$ is calculated as:

$$C_U = \frac{d_{50}}{d_{10}} = \frac{0.121}{0.103} = 1.17$$
APPENDIX B

TIME HISTORIES
B.1 Test-1 Time Histories

Figure B-1. Test-1 acceleration time histories at 0.1g shaking step

Figure B-2. Test-1 acceleration time histories at 0.2g shaking step
Figure B-3. Test-1 acceleration time histories at 0.3g shaking step

Figure B-4. Test-1 acceleration time histories at 0.4g shaking step
Figure B-5. Test-1 acceleration time histories at 0.5g shaking step

Figure B-6. Test-1 acceleration time histories at 0.6g shaking step
Figure B-7. Test-1 displacement time histories at 0.1g shaking step

Figure B-8. Test-1 displacement time histories at 0.2g shaking step
Figure B-9. Test-1 displacement time histories at 0.3g shaking step

Figure B-10. Test-1 displacement time histories at 0.4g shaking step
Figure B-11. Test-1 displacement time histories at 0.5g shaking step

Figure B-12. Test-1 displacement time histories at 0.6g shaking step
B.2 Test-2 Time Histories

Figure B-13. Test-2 acceleration time histories at 0.3g shaking step
Figure B-14. Test-2 acceleration time histories at 0.4g shaking step
Figure B-15. Test-2 acceleration time histories at 0.5g shaking step
Figure B-16. Test-2 displacement time histories at 0.3g shaking step

Figure B-17. Test-2 displacement time histories at 0.4g shaking step
Figure B-18. Test-2 displacement time histories at 0.5g shaking step
B.3 Test-3 Time Histories

Figure B-19. Test-3 acceleration time histories at 0.6g, 10 Hz frequency shaking step
Figure B-20. Test-3 acceleration time histories at 0.4g, 5Hz shaking step, as per standard protocol.
Figure B-21. Test-3 acceleration time histories at 0.5g shaking step
Figure B-22. Test-3 acceleration time histories at 0.6g shaking step
Figure B-23. Test-3 acceleration time histories at 0.7g shaking step
Figure B-24. Test-3 displacement time histories at 0.6g, 10 Hz frequency shaking step

Figure B-25. Test-3 displacement time histories at 0.4g shaking step

XXV
Figure B-26. Test-3 displacement time histories at 0.5g shaking step

Figure B-27. Test-3 displacement time histories at 0.6g shaking step

XXVI
Figure B-28. Test-3 displacement time histories at 0.7g shaking step.
B.4 Test-4 Time Histories

Figure B-29. Test-4 acceleration time histories at 0.1g shaking step
Figure B-30. Test 4 acceleration time histories at 0.2g shaking step
Figure B-31. Test-4 acceleration time histories at 0.3g shaking step

XXX
Figure R-32. Test-4 acceleration time histories at 0.4g shaking step
Figure B-33. Test-4 acceleration time histories at 0.5g shaking step
Figure B-34. Test-4 acceleration time histories at 0.6g shaking step
Figure B-35. Test-4 acceleration time histories at 0.65g shaking step
Figure B-36. Test-4 displacement time histories at 0.1g shaking step

Figure B-37. Test-4 displacement time histories at 0.2g shaking step

XXXV
Figure B-38. Test-4 displacement time histories at 0.3g shaking step

Figure B-39. Test-4 displacement time histories at 0.4g shaking step
Figure B-40. Test-4 displacement time histories at 0.5g shaking step

Figure B-41. Test-4 displacement time histories at 0.6g shaking step
Figure B-42. Test-4 displacement time histories at 0.65g shaking step
Figure B-43 Test-5 acceleration time histories at 0.1g shaking step.
Figure B-44. Test 5 acceleration time histories at 0.2g shaking step.
Figure B-45. Test-5 acceleration time histories at 0.3g shaking step.
Figure B-46. Test-5 acceleration time histories at 0.4g shaking step.
Figure B-47. Test 5 acceleration time histories at 0.5g shaking step.
Figure B-48. Test-5 acceleration time histories at 0.6g shaking step.
Figure R.49. Test-5 acceleration time histories at 0.7g shaking step.
Figure B-50. Test-5 displacement time histories at 0.1g shaking step

Figure B-51. Test-5 displacement time histories at 0.2g shaking step
Figure B-52. Test-5 displacement time histories at 0.3g shaking step

Figure B-53. Test-5 displacement time histories at 0.4g shaking step
Figure B-54. Test-5 displacement time histories at 0.5g shaking step

Figure B-55. Test-5 displacement time histories at 0.6g shaking step
Figure B-56. Test-5 displacement time histories at 0.7g shaking step
B.6 Test-6 Time Histories

Figure B-57. Test-6 acceleration time histories at 0.1g shaking step.
Figure B-58. Test-6 acceleration time histories at 0.2g shaking step.
Figure B-59. Test-6 acceleration time histories at 0.3g shaking step.
Figure B-60. Test-6 acceleration time histories at 0.4g shaking step.
Figure B-61. Test-6 acceleration time histories at 0.5g shaking step.
Figure B-62. Test-6 displacement transducer time histories at 0.5g shaking step.

Figure B-63. Test-6 displacement time histories at 0.2g shaking step.
Figure B-64. Test-6 displacement time histories at 0.3g shaking step.

Figure B-65. Test-6 displacement time histories at 0.4g shaking step.
Figure B-66. Test-6 displacement time histories at 0.5g shaking step.
B.7 Test-7 Time Histories

Figure B-67. Test-7 acceleration time histories at 0.1g shaking step.
Figure D-68. Test-7 acceleration time histories at 0.2g shaking step.
Figure B-69. Test-7 acceleration time histories at 0.3g shaking step.
Figure B-70. Test-7 acceleration time histories at 0.4g shaking step.
Figure B-71. Test-7 acceleration time histories at 0.5g shaking step.
Figure B-72. Test-7 acceleration time histories at 0.6g shaking step.
Figure B-73. Test-7 acceleration time histories at 0.7g shaking step.
Figure B-74. Test-7 displacement transducer time histories at 0.1g shaking step.

Figure B-75. Test-7 displacement transducer time histories at 0.2g shaking step.
Figure B-76. Test-7 displacement time histories at 0.3g shaking step.

Figure B-77. Test-7 displacement time histories at 0.4g shaking step.
Figure B-78. Test-7 displacement time histories at 0.5g shaking step.

Figure B-79. Test-7 displacement time histories at 0.6g shaking step.
Figure B-80. Test-7 displacement time histories at 0.7g shaking step.
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B.2  Test-2 Time Histories ............................................................ XV
B.3  Test-3 Time Histories ............................................................ XX
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B.5  Test-5 Time Histories ............................................................ XXXIX
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B.7  Test-7 Time Histories ............................................................. LVIII

Figure B-1. Test-1 acceleration time histories at 0.1g shaking step ........................ IX
Figure B-2. Test-1 acceleration time histories at 0.2g shaking step ........................ IX
Figure B-3. Test-1 acceleration time histories at 0.3g shaking step ....................... X
Figure B-4. Test-1 acceleration time histories at 0.4g shaking step ....................... X
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C.2 Test-3 Testing Images

Figure C-5. Test-2 prior to testing.

Figure C-6. Test-3 after first shaking step of 0.6g, at 10Hz (due to experimental error).

Figure C-7. Test-3 after 0.4g, 5 Hz shaking step (standard testing protocol resumed).
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Figure C-14. Test-4 after 0.3g shaking step.

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C.4 Test-5 Testing Images

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C.6 Test-7 Testing Images

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Figure C-28. Test-7 after 0.1g shaking step.

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Figure C-4. Test-2 after 0.5g shaking.

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Figure C-13. Test-4 after 0.2g shaking step.

Figure C-14. Test-4 after 0.3g shaking step.

Figure C-15. Test-4 after 0.4g shaking step.

Figure C-16. Test-4 after 0.5g shaking step.

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Figure C-19. Test-5 prior to testing.

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Figure C-21. Test-5 after 0.2g shaking step

Figure C-22. Test-5 after 0.3g shaking step

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Figure C-26. Test-5 after 0.7g shaking step

Figure C-27. Test-7 prior to testing.

Figure C-28. Test-7 after 0.1g shaking step.

Figure C-29. Test-7 after 0.2g shaking step.
Figure C-30. Test-7 after 0.3g shaking step.  LXXXII

Figure C-31. Test-7 after 0.4g shaking step.  LXXXII

Figure C-32. Test-7 after 0.5g shaking step.  LXXXII

Figure C-33. Test-7 after 0.6g shaking step.  LXXXIII

Figure C-34. Test-7 after 0.7g shaking step.  LXXXIII
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GEOPIV PLOTS OF MAXIMUM SHEAR STRAIN
D.1 Test-6 Residual Maximum Shear Strain

Figure D-1. Maximum shear strain within the reinforced soil region for Test-6, vertical wall reinforced at L/H = 0.6, by shaking steps: a) 0.1g, b) 0.2g, c) 0.3g, d) 0.4g.
D.2 Test-5 Residual Maximum Shear Strain

a) 0.1g

b) 0.2g

c) 0.3g

d) 0.4g

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Figure D-2. Maximum shear strain within the reinforced soil region for Test-5, vertical wall reinforced at L/H = 0.9, by shaking steps: a) 0.1g, b) 0.2g, c) 0.3g, d) 0.4g, e) 0.5g, and f) 0.6g.

D.3 Test-5 Maximum Shear Strain developed during 0.7g shaking step
Figure D-3. Development of maximum shear strain within the reinforced soil region during 0.7g shaking step for Test-5, vertical wall reinforced at L/H = 0.9. Strains accumulated from the end of 0.6g shaking step to: a) 10 cycles, b) 20 cycles, c) 30 cycles, d) 40 cycles, and e) 50 cycles.
D.4 Test-7 Residual Maximum Shear Strain

a) 0.1g

b) 0.2g

c) 0.3g

d) 0.4g

e) 0.5g

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Figure D-4. Maximum shear strain within the reinforced soil region for Test-7, wall reinforced at L/H = 0.9 and inclined at 70° to the horizontal, by shaking steps: a) 0.1g, b) 0.2g, c) 0.3g, d) 0.4g, e) 0.5g, and f) 0.6g.

D.5 Test-7 Maximum Shear Strain developed during 0.7g shaking step
Figure D-5. Development of maximum shear strain within the reinforced soil region during 0.7g shaking step for Test-7, wall inclined at 70° to the horizontal and reinforced L/H = 0.75. Strains accumulated from the end of 0.6g shaking step to: a) 10 cycles (c), b) 20 cycles, c) 30 cycles, d) 40 cycles, and e) 50 cycles.