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Centrifuge Testing of a Sheet Pile Wall with Clay Backfill

S. P. Gopal Madabhushi* and V. S. Chandrasekaran**

Introduction

Sheet pile walls are a common form of earth retaining structures. They are used for temporary as well as permanent earth retention of highway and railway cuttings, at bridge abutments, in jetties in harbours etc. In Geotechnical Practice, when the height of the earth that needs to be retained is rather high, the sheet piles walls are usually anchored near the top. On the other hand when this height is small, cantilever sheet pile walls are employed. In this paper the emphasis will be on the later type where cantilever sheet pile walls are used to retain a moderate height of earth.

While sheet pile walls both cantilever type and anchored bulk head type have been used for many decades, they often provide a challenge to geotechnical engineers to predict their behaviour in terms of deformations and the earth pressures sustained by them. In this paper physical modelling in the form of centrifuge testing will be used to establish the failure mechanisms suffered by the sheet pile walls. The fact that the sheet pile wall problem is well researched and well understood would be used in this paper to establish the efficacy of the modelling technique used. In this paper it will also be shown how the centrifuge testing can add to the understanding of the behaviour of the sheet pile walls.

The cantilever sheet pile retaining walls are interesting in that they have close interaction with the earth they retain. The backfill material that constitutes the earth retained by the sheet pile wall plays an important role in determining the behaviour of these structures. One usually employs Rankine or Coulomb earth pressure theories to predict the earth pressures exerted by the soil on the sheet pile wall. Wroth (1972) discusses the earth pressure theories in detail for granular backfills. It is also well known (for example Bolton, 1979 or Broms, 1995) that a flexible retaining wall needs to suffer a certain amount of deformation so that the requisite strains are mobilised in the soil and the active and passive earth pressure regimes prevail. Most Soil Mechanics text books introduce the topic of earth pressures and give the routine design methodology

* Reader and Asst Director (Schofield Centre), Department of Engineering, University of Cambridge, UK, (also Adjunct Associate Professor, Department of Civil Engineering, Indian Institute of Technology, Bombay and on sabbatical leave from University of Cambridge, UK), Email: mspg1@eng.cam.ac.uk

** Formerly Professor, Department of Civil Engineering, Indian Institute of Technology, Bombay, Email: vsc03@yahoo.co.in

of sheet pile walls adopted in the Geotechnical Practice. For example Craig (1987) explains the active and passive earth pressures generated when the backfill material is granular or cohesive. He advises the reader to avoid use of cohesive material as backfill, if at all possible. This may not however be entirely possible in the field as in many instances the locally available backfill material may be fine grained and many of the walls are constructed by excavation.

The active earth pressure exerted by a horizontal, cohesive backfill with no surcharge is given by

$$\sigma_a = \gamma z - 2c_u \tag{1}$$

where γ is the unit weight of the soil, z is the depth of the soil and c_u is the undrained shear strength of the soil. Similarly the passive earth pressure may be expressed as;

$$\sigma_p = \gamma z + 2c_u \tag{2}$$

For backfills with cohesive soils the failure mechanism may be expected to be as outlined in Figure 1. The retaining wall is expected to rotate about a point below the excavation level as shown in this figure. The corresponding earth pressure distribution is presented in Figure 2. The active earth pressure distribution is as per equation 1 except that the tensile stress near the top is ignored. The net earth pressure below the excavation level and above the point of rotation is taken to be '4 c_u - γH' and the net earth pressure below the point of rotation is taken as '4 c_u + γH'.

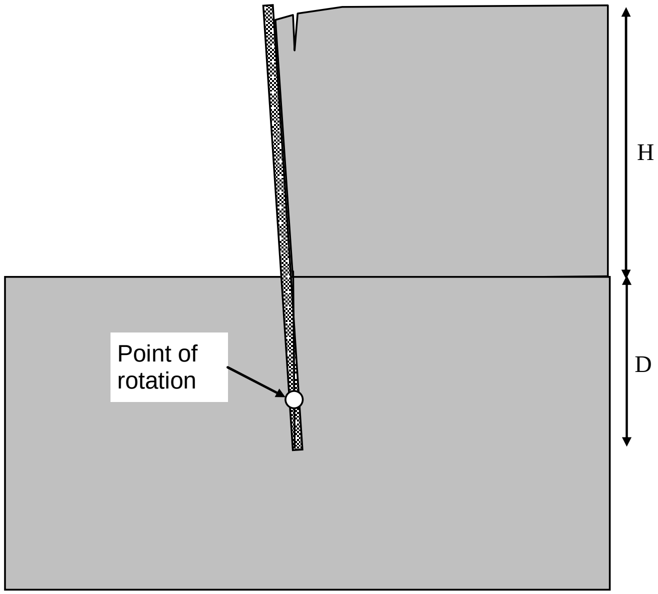


Fig. 1 Failure of a Sheet Pile Wall with Cohesive Backfill

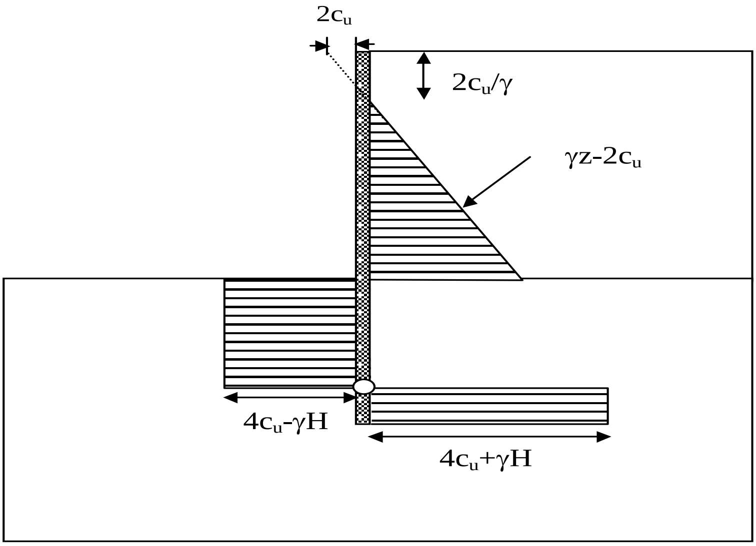


Fig. 2 Net Earth Pressures Acting on the Sheet Pile Wall with Clayey Backfill

Behaviour of sheet pile walls is a well researched area in geotechnical engineering. Terzaghi (1934a,b) carried out some of the early tests on large scale model retaining walls that were hinged at the base and this work formed the basis for much of the subsequent research. Bransby and Milligan (1975) have carried out laboratory scale experiments on retaining walls and showed that it is possible to link the wall deformations to the soil deformations. Padfield and Mair (1984) outlined the design method used for sheet pile walls for both cohesionless and cohesive backfill materials in the UK. Bolton and Powrie (1987 and 1988) have considered both cantilever and anchored diaphragm walls and report a series of centrifuge tests with such walls in over-consolidated clay. In these experiments they investigated the influence of various parameters such as the stress history of the soil, over-consolidation ratio and height/depth ratio of the retaining walls. In these centrifuge tests the construction process of diaphragm walls was simulated by using heavy fluid placed in rubber bags in front of the model walls that exerted the correct horizontal pressure on the wall which was equivalent to the horizontal earth pressure prior to excavation. Pumping this heavy fluid out of the rubber bag simulated the excavation process and the resulting wall displacements and bending moments generated in the wall section were measured. King (1995) also carried out centrifuge modelling of cantilever sheet pile walls with cohesionless backfills and recommended that the pivot point about which the walls rotate may be determined using centrifuge model tests. Bica and Clayton (1998) described laboratory based 1-g experiments on cantilever sheet pile walls and showed that the earth pressures below the pivot point were smaller than Rankine passive earth pressure as the wall friction acted downwards below the pivot point. Madabhushi and Chandrasekaran (2005) proposed a minimisation of moment ratio technique to determine the point of rotation for retaining walls with cohesive and cohesionless backfills.

In this paper, the behaviour of cantilever retaining walls with cohesive soils forming the backfill is considered. Results from centrifuge tests that were carried out to elucidate the failure mechanisms of cantilever sheet pile walls retaining such cohesive backfill material will be presented. It must be pointed out that the stress history of the soil due to the excavation process has not been modelled in this study. Instead the focus of this paper is on establishing the use of centrifuge modelling technique in solving practical problems. More advanced models with layered backfill soils or more complicated stress histories can be simulated on centrifuge models in-flight as explained above, to simulate more accurately the actual field problems.

Soil Characterisation

The physical scaled models tested in the centrifuge need to be made from the same material that is encountered in the field. The soil used in this series of centrifuge tests was Bombay Marine clay. The properties of this clay are well known and are presented in Table 1. This clay was used in several previous studies at IIT Bombay, for example Katti et al (1985).

Table 1 Properties of Bombay Marine Clay

Physical Properties	*Value*
Liquid Limit	75%
Plastic Limit	43%
Plasticity Index	32%
Shrinkage Limit	17.3%
Specific Gravity	2.76
Optimum Moisture Content	35%
Maximum Dry Density	1300 kg/m^3
Textural Composition	
Sand (<2mm and > 0.06mm)	2.4 %
Silt (<0.06mm and > 0.002mm)	32%
Clay (< 0.002 mm)	65.6%
Textural classification	CH
Classification	Clay

after Katti et al (1985)

The required quantity of the Bombay Marine Clay was air dried and sieved to remove material coarser than 2 mm. This clay was then mixed with water to give a nominal water content of 40%. This is 5% more than the optimum moisture content. The actual water content was determined by taking random samples from the soil model. This revealed that the actual water content of the soil model was 53.8%. This may be attributed to some initial moisture that remained in the air dried clay and the high relative humidity in Bombay resulting in high hygroscopic moisture.

The soil was placed in the model container in layers of about 30 mm thickness and each layer was initially subjected to light compaction. The Standard Proctor compactor was then used and each layer was subjected to about 25 blows distributed uniformly on the soil sample. Soil samples were taken from the compacted layers and Unconfined Compressive Strength (UCS) tests were carried out to determine the undrained shear strength c_u of the clay in the soil model. The stress-strain curves observed during the UCS tests are presented in Figure 3. In this figure we can see that the unconfined compressive strength q_u is between 60 to 65 kPa. The clay may be classified as soft clay according to BS 8004:1986 (See Craig ,1987)

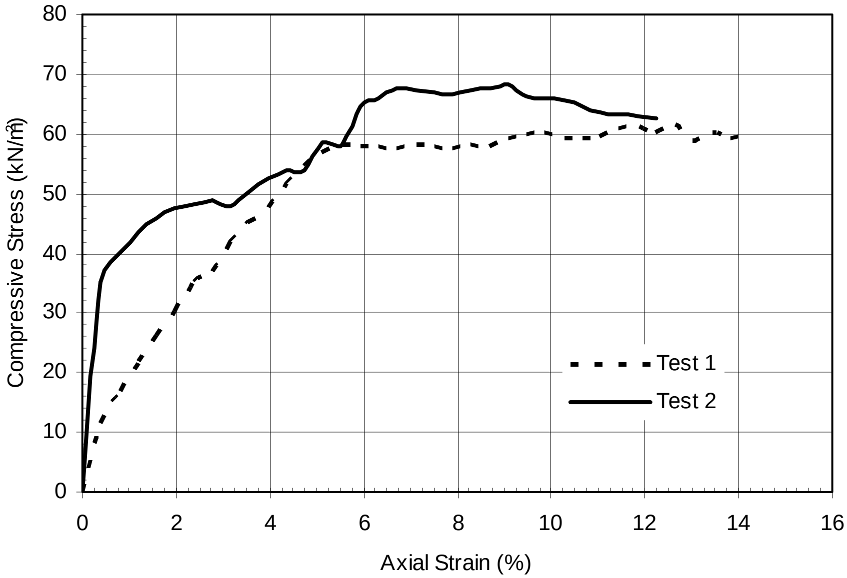


Fig. 3 Stress-Strain Curves from the Soil Samples during UCS Tests

Model Preparation and Testing Procedure

The strong box was 760 mm long 200 mm wide and 410 mm deep. The dimensions of the model and its positioning in the container are is shown in Figure 4. It may be seen there is sufficient space for the development of active and passive wedges. Model dimensions for all the centrifuge tests reported in this paper were kept the same except for the penetration depth of the retaining wall below excavation level. In the first centrifuge test the penetration depth was 130mm below excavation level, while in the second centrifuge test this was 75 mm below the excavation level. Thus the penetration depth to the height retained ratio (D/H) in these two tests was 0.76 and 0.44 respectively.

The soil was placed in layers of about 30 mm thickness and was hand kneaded into a level surface. The layer was then subjected to compaction as described above. White marker lines of very fine white silica sand were placed next to the Perspex window of the model container approximately at 60 mm intervals. The bulk density of the soil in each layer was determined by weighing the soil that was used to make each layer and the volume of the compacted layer. The average bulk density of the compacted clay was found to be about 1685 kg/m^3.

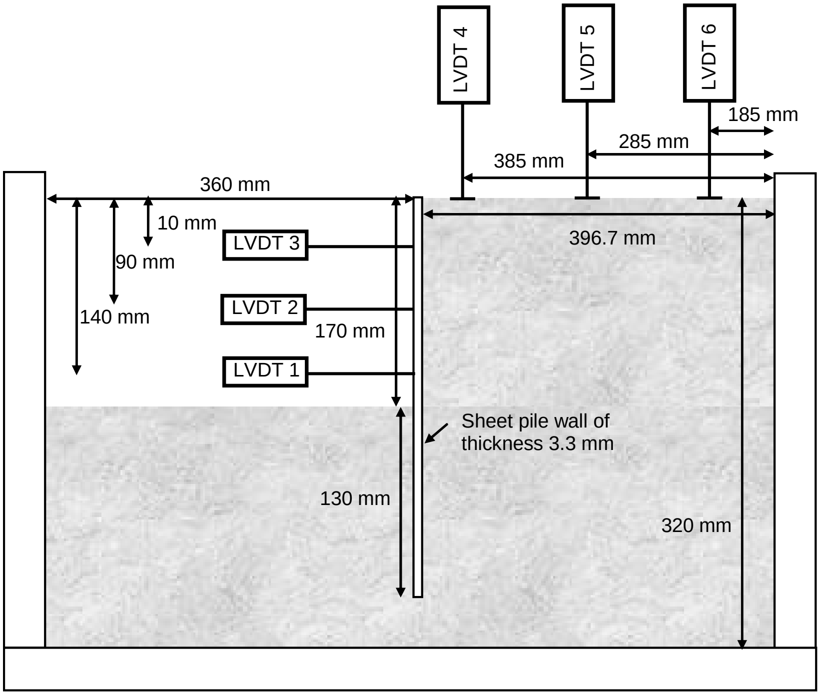
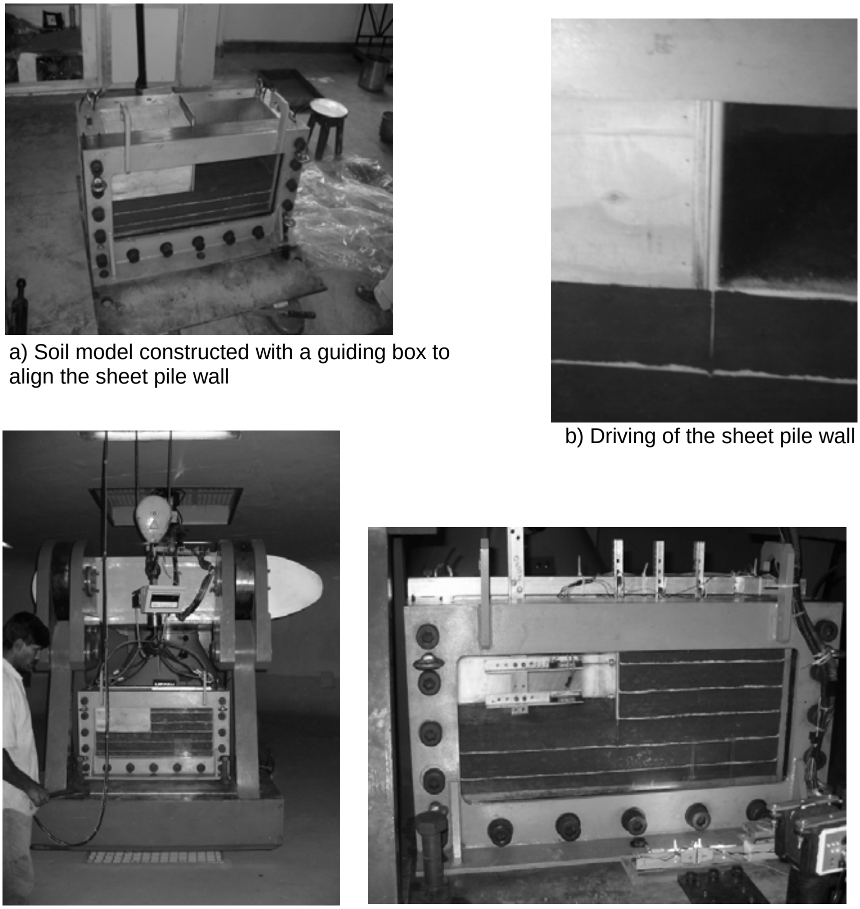


Fig. 4 A Schematic Diagram of the Cross-Section of Centrifuge Model SPGM-1

The model sheet pile was made of a 3.3 mm thick Dural (Aluminium alloy) sheet. The properties of this material and equivalent flexural stiffness of a prototype sheet pile wall are presented in Table 2. The base of the sheet pile wall was tapered to facilitate easy driving in. The soil model was constructed to the height of 150mm and the sheet pile wall was driven into place. A supporting wooden box was made to guide the sheet pile wall on one side as seen in Figure 5a. The sheet pile was driven into place as seen in Figure 5b. The guiding wooden box remained in place until the model was fully prepared and loaded on to the centrifuge swing basket as seen in Figure 5c. After the sheet pile was driven to the required depth, the backfill side of the wall was constructed in layers as described above. Marker lines are placed at regular intervals. The supporting wooden box was removed just prior to the flight. After the wooden box was removed, LVDT's were placed to measure the vertical settlements of the backfill as well as the lateral deflection of the sheet pile wall. In all six LVDT's were used , 3 for vertical settlement measurement and 3 for lateral deflection measurement. The LVDT's and the support gantry fixed onto the model container can be seen in Figure 5d. LVDT's 1 to 4 had a stroke of 50mm while 5 and 6 had stroke of 25mm.

Table 2 Material properties of the model sheet pile wall

Parameter	*Value*
Thickness	3.3 mm
Width	198 mm
Height	
Test SPGM-1	300 mm
Test SPGM-2	245 mm
Young's Modulus	70 GPa
Density	2830 kg/m^3
Flexural Stiffness (EI) of model sheet pile wall	209.63 Nm^2/m
Flexural Stiffness (EI) an equivalent prototype sheet pile wall at 50g*	1310.20 MNm^2/m



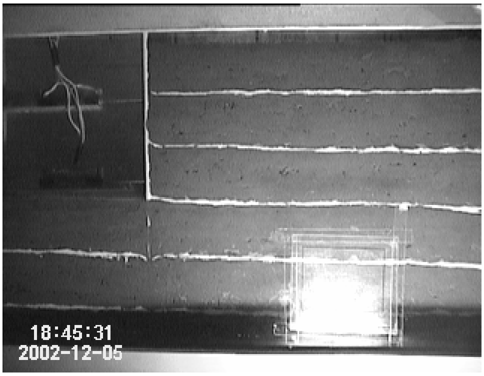
a) Soil model constructed with a guiding box to align the sheet pile wall

b) Driving of the sheet pile wall

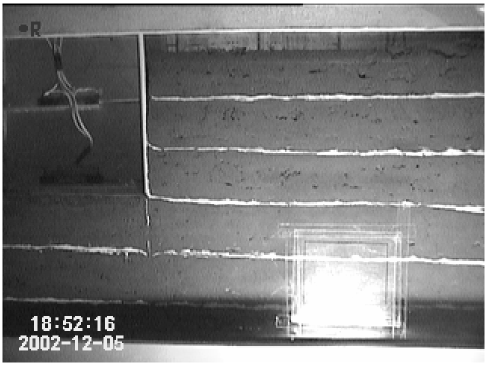
c) Loading of the container on the wing basket of the centrifuge

d) A view of the model with LVDT's and flight camera in place

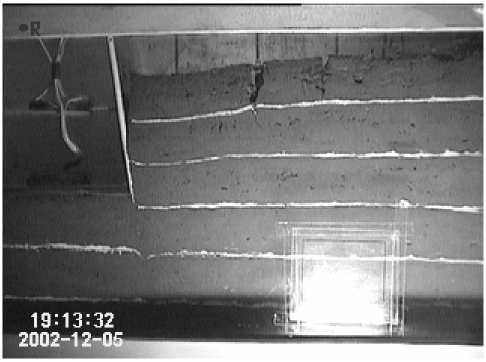
Fig. 5 Placement of Model on the Centrifuge



a) Centrifuge model at 10g



b) Centrifuge model at 30g



c) Centrifuge model at 50g

Fig. 6 In-Flight Photographs of the Centrifuge Model SPGM-1 Taken at Various g Levels

The mass of the model container with the soil was determined carefully along with the centroidal heights. The typical mass of the model container and the soil used was about 520 kg. At 50g this would exert a radial force of 255.06 kN (25.5 tons) on the centrifuge. This force needs to be balanced by the radial force generated by the counter weight. This information is necessary to adjust the counter weight of the centrifuge so that the net force along the central axis of the centrifuge is zero. Pre-flight checks were carried out and the centrifuge was started. In all tests reported in this paper, the procedure was to increase the centrifugal acceleration in stages to 10g, 20g, 30g, 40g and 50g. Each g level was maintained for only a short duration of about 2.5 minutes so that there is very little time for consolidation of the soil to take place. As a result the clay can be assumed to be in an undrained state. Through out the centrifuge test, the settlement of the backfill and lateral deflection of the sheet pile wall were monitored continuously by logging the data from the LVDT's. At each g level in-flight photographs were taken of the model using the on-board camera. In Figure 6 we present a typical set of such photographs taken at 10g, 30g and 50g. At 10g we can see that the deformations of the model are modest. At 30g we can see that the deformations are still modest, but a clear vertical crack has opened up between the model sheet pile wall and the backfill to a depth of 80mm (each layer between the marker lines is approximately 60 mm). At 50g the sheet pile wall has suffered excessive lateral deflection. Tension cracks appear in the backfill at three different locations. The depth of the large tension cracks can be estimated to be about 90mm below the soil surface with minor cracks extending to a depth of 110mm. It must be pointed out that the prototype represented by the centrifuge model is changing with the g level. The prototype heights of retained soil at 30g and 50g will be 5.1m and 8.5m respectively.

After the wall suffered severe deformations, the centrifuge was stopped. The model was then subjected to a post-test investigation. The deformed shape of the wall and the backfill were measured carefully. Also digital images were taken of the model. Soil samples were taken by driving sampling tubes at selected locations on the backfill side and near the toe of the sheet pile wall. UCS tests were carried out on the samples.

In the case of the second centrifuge test SPGM-2 this procedure was slightly modified. The first flight was as described above. But once the first flight was completed, it was observed that the model sheet pile wall was deriving support from the LVDT's as they reached the end of their travel. A further flight was carried out in which the LVDT's and their support gantries were removed and the model was flown to 50g. This led to further rotation of the model sheet pile wall and the development of passive wedges at the toe of the wall. This will be discussed in next section.

Deformations

In this section we shall consider the deformations observed in the soil and the lateral displacements suffered by the sheet pile wall in both these centrifuge tests.

Centrifuge Test SPGM-1

As mentioned before the ratio of penetration depth to the height of soil retained (D/H) for this centrifuge model was 0.76. Also there is a 20mm layer of soil below the sheet pile wall before the base boundary is reached. During the centrifuge test the model sheet pile wall suffered severe lateral displacements. In Figure 7 the lateral displacements recorded by LVDT's 1 to 3 at various g levels are presented. In Figure 4 the location of all the LVDT's is shown.

It can be seen from Figure 7 that the ultimate lateral displacements suffered by the wall were excessive. Initially up to about 20g the lateral displacements are small. The top of the wall displaced by about 3 mm as recorded by LVDT3 as the g level was increased from 20g to 30g. As the g level was increased further to 40g the top of the wall displaced by about 10mm. This would represent a displacement of 400mm in a corresponding prototype. Further as the g level was increased from 40g to 50g, the top of the wall did not displace in a continuous fashion. It saw a large increase in lateral displacement from 10mm to 32mm and then stopped. A further lateral displacement of about 3mm occurred before the 50g was reached. While this g level was maintained the wall suffered a further 7 mm displacement taking the total lateral displacement to about 42mm. This would represent a prototype lateral displacement of about 2.1m in a sheet pile that is retaining 8.5m of earth.

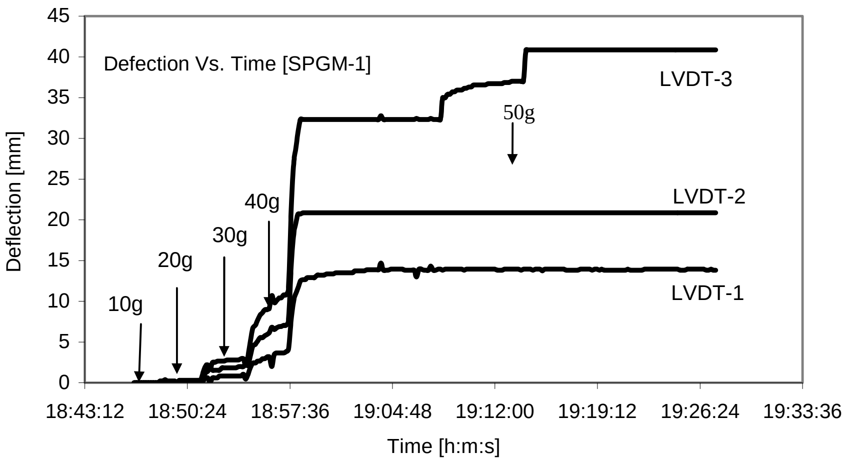


Fig. 7 Lateral Deflection of the Sheet Pile Wall at Various g Levels

In Figure 8 the rotation suffered by the sheet pile wall at different times is presented. The rotation of the wall is calculated using the readings of LVDT's 1 and 2 and knowing the distance between the two LVDT's. Again it can be seen that the wall rotation increased with increasing g level in a discontinuous fashion. At 50g the rotation reached a staggering 12^o to the vertical.

In a sheet pile wall retaining soft clay active and passive pressures may be expected to mobilise when the top displacement of the wall reaches 5% of the height of retained earth, Azizi (2000). In the current experiments it may be seen that the above top displacements were reached at a g level just above 35g. This represents a wall height of about 6m. For walls below this height there is a cohesion demand (minimum cohesion required for stability) that is less than the undrained shear strength , whereas for walls above this height the cohesion demand is more and the soil shear strength is not adequate to maintain stability of the wall. This is well supported by the present experiments.

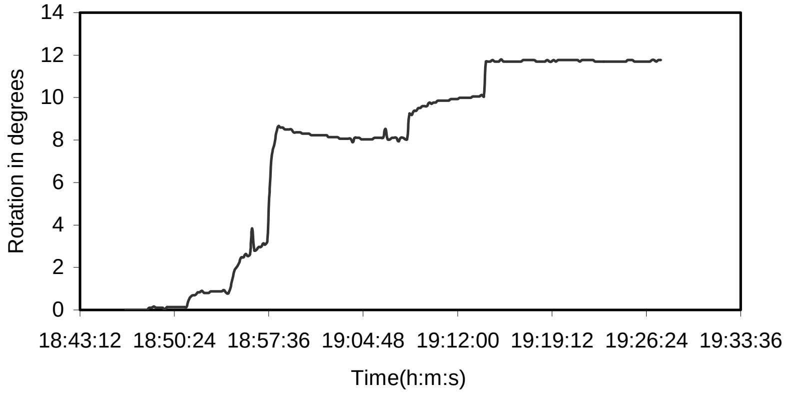


Fig. 8 Variation of the Rotation of the Top of the Model Sheet Ppile Wall with Time

It is also interesting to see the discontinuous fashion in which the top of the wall suffers the lateral displacements. This may be due to the formation of the tension cracks in the backfill soil. Up to 30g there were no tension cracks visible as seen in Figure 6. As the g level was increased to 40g and then to 50g large tension cracks appeared. As the g level increased the vertical stress in the soil increases thereby causing an increase in the horizontal stress. The wall reacts to this and subjects the soil on the passive side to increased horizontal stress. The soil in this region starts to 'bulge out' allowing for the wall to suffer rotation. The system would come to equilibrium at this stage. However, when a tension crack is initiated there is a further increase in the horizontal stress that would lead to a further rotation of the wall. This corroborates that the discontinuous increase in wall displacement that will be governed by both increase of the g level and initiation of tension cracks. Each time a tension crack is formed the wall will suffer a further rotation until equilibrium is achieved.

The settlement of the backfill occurs as the g level is increased. In Figure 9, the settlements recorded by the LVDT's 4 to 6 are presented. The location of these devices can be seen in Fig.4.

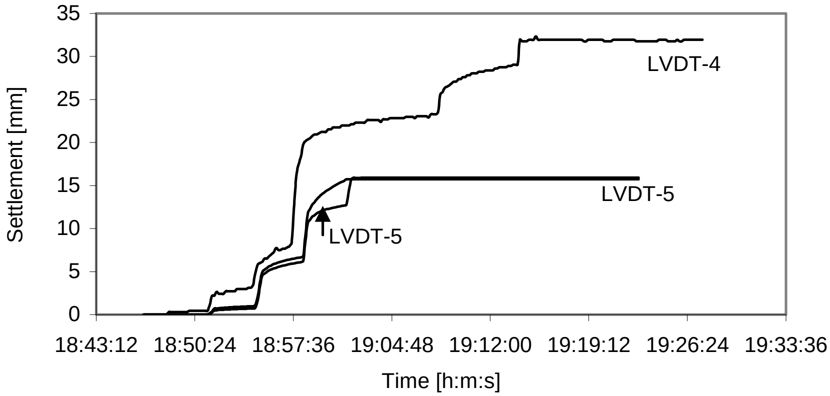


Fig. 9 Settlement of the Backfill with Increase in g Level

LVDT 4, which is closest to the sheet pile wall, records the maximum amount of settlement. At 50g this value reached 32 mm and this would correspond to a prototype settlement of 1.6m. As mentioned before LVDT's 5 and 6 had a shorter stroke and therefore could not respond after the settlement at their locations reached about 20mm. These devices were unable to record any further settlements that were taking place. Again the settlement behind the wall was taking place in a discontinuous fashion, similar to the lateral displacements. This also confirms that the initiation of the tension cracks play a major role on the timing of the deformations.

Post-test investigation was carried out on the model after the centrifuge test. In Figure 10 we present the overview of the model after the Perspex window was removed. The large deformations suffered by the model sheet pile wall and the formation of the tension cracks in the backfill are clearly visible in this figure.

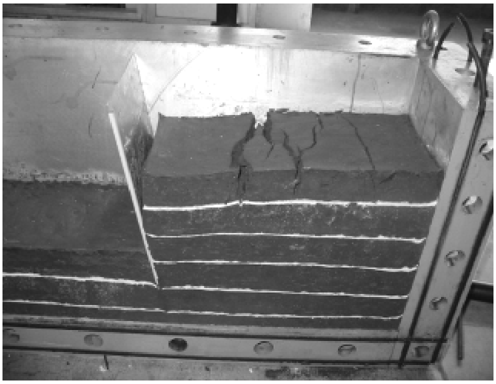


Fig. 10 A View of the Model SPGM-1 Post Flight Showing the Rotation of the Model Sheet Pile Wall and the Tension Cracks in the Backfill

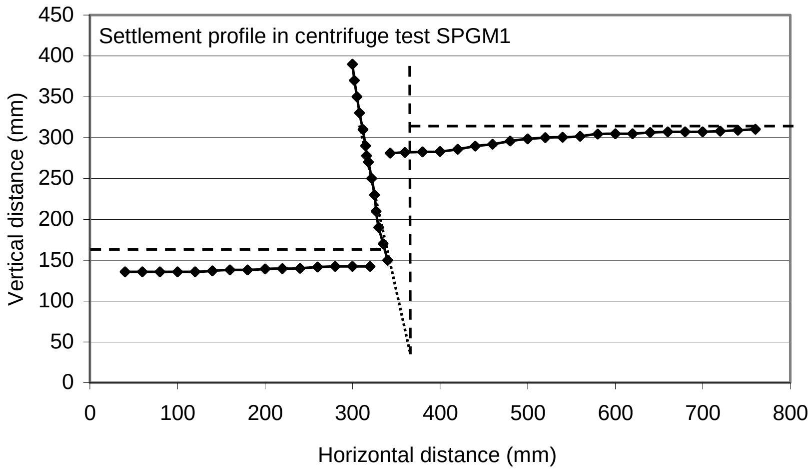


Fig.11 Measured Settlement and Heave of Clay and the Rotation of the Model Sheet Pile Wall After Centrifuge Test SPGM -1

The surface profile of the soil and the lateral deflection of the wall were carefully measured to verify whether they corroborated with the in-flight measurements logged by the LVDT's. This is presented in Figure 11 in which the original ground levels and the position of the sheet pile wall are indicated as dashed lines. In this figure we can see the large settlements that have taken place on the backfill side and also the heave that has occurred on the front side of the wall. The sheet pile wall itself did not suffer any bending, but rotated as a rigid body as seen in Figure 11. An additional straight line is added in this figure, which extends the measured line of sheet pile wall into the soil. As indicated by this line, we can see that the point of rotation is about a point close to and above the base of the sheet pile wall.

Centrifuge test SPGM-2

Following the first centrifuge test it was decided to investigate the effect of depth of penetration of the sheet pile wall on the failure mechanism. It is possible that if the sheet pile wall is driven to insufficient depth , the passive resistance in the embedded portion of the wall may be overcome. In this centrifuge test it was decided to investigate if such a failure mechanism could be induced in which the toe of the wall moves laterally with substantial soil heave at the toe. Consequently the ratio of penetration depth to the height of soil retained (D/H) for this centrifuge model was reduced to 0.44.

As discussed before this centrifuge test was carried out in two flights. The first flight involved taking the model to 50g and monitoring the wall deflection and soil settlements as in centrifuge test SPGM-1. In the second flight, the LVDT's and their support gantries were removed so that the sheet pile wall cannot derive any support from these.

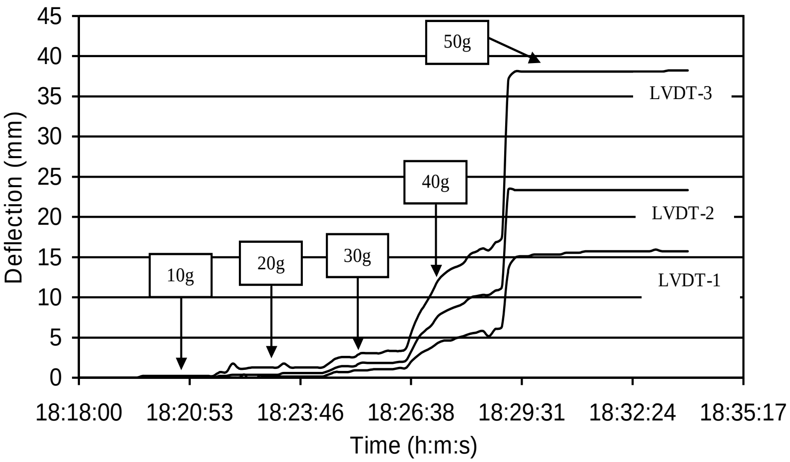


Fig.12 Lateral Deflections of the Sheet Pile Wall at Various g Levels in Centrifuge Test SPGM-2

The data acquired by the LVDT's 1 to 3 is presented first. In Figure 12 the lateral deflections underwent by the sheet pile wall are seen. Clearly the LVDT 3, closest to the top of the sheet pile wall records the maximum deflection. As in the previous centrifuge test at lower g levels up to 30g, there is an incremental increase in the lateral displacement but the sheet pile wall came into equilibrium. However at higher g levels this was not the case and sheet pile wall suffered steady increase in the lateral displacement. As the g level was being increased from 40g to 50g there was a sudden failure and all the LVDT's reached their stroke limits. In this test, LVDT's 1 and 2 had a stroke of about 25mm. This would be equivalent to a 'catastrophic failure' in the field where a sheet pile wall fails suddenly. This did not happen in the earlier centrifuge test where the depth of penetration of the sheet pile wall was more. As before it is possible to plot the rotation data of the wall and the settlement of the back fill soil. These will be similar to previous centrifuge test and therefore are not presented in this paper for brevity

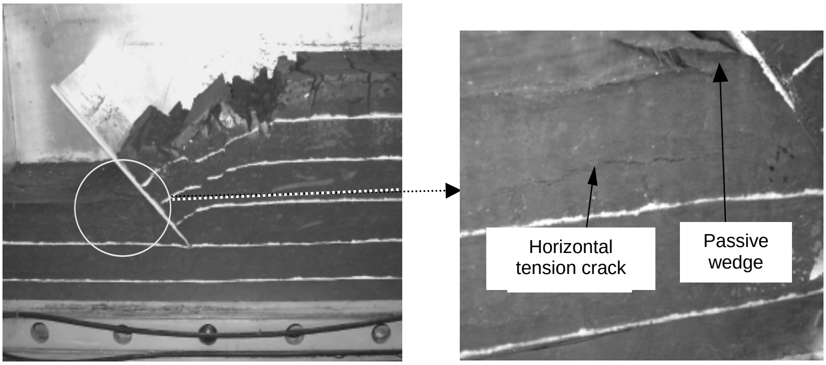
The interesting aspect of the model behaviour was in the second flight. In this flight, the LVDT support gantries do not support the model sheet pile wall. As a result the sheet pile wall could suffer further rotation. In-flight pictures presented in Figure 13 show the soil deformations have increased substantially as the g level was increased from 40g to 50g. The rotation of the wall has increased substantially confirming that the LVDT gantries in the previous flight offered support beyond 40g. It is interesting to note that the point of rotation of the wall can be seen in Figure 13 to be at a point close to the base of the sheet pile wall. So despite the depth of penetration was reduced to 0.44 in this test, the sheet pile wall has still rotated about a point close to and above the base.

a) Centrifuge Model at 40g b) Centrifuge Model at 50g

Fig.13 In-Flight Photographs of Centrifuge Model SPGM-2 at Various G Levels in Flight 2

After the second flight, post-test investigations were carried out on the centrifuge model. In Figure 14 the views of the model showing the large deformations of the soil and the tension cracks in the backfill are presented. In Figure 14a it may be seen that the backfill has suffered severe deformations owing to the excessive rotation of the wall. In this figure it can also be seen that there are significant strains suffered by the backfill soil normal to the marker lines. Given that the marker lines were placed 60mm apart, it was measured that the normal compressive strains of about 50% have occurred. As the soil is in an undrained state, these normal compressive strains will be translated as tensile strains tangential to the marker lines, in order to maintain the 'no volume change' condition. These tensile strains allowed for the large soil deformations that needed to occur once the sheet pile wall suffered severe rotation.

A close up view of the passive wedge that has formed is presented in Figure 14b. The size of the passive wedge is fairly small. It is also interesting to note that a horizontal tension crack has appeared in the passive zone and this may be seen in the figure.



a) Sheet pile wall rotation   b) Formation of passive wedge

Fig. 14 Views of the Centrifuge Model SPGM-2 Post-Test, Showing the Large Deformations, Formation of Tension Cracks and the Passive Wedge

In Figure 15 the settlement profile of the soil measured post-test is presented. In this figure the excessive settlement of the backfill and the heave that has occurred in the passive region can be seen. Also the rotation suffered by the wall was measured. As seen in the previous centrifuge test the sheet pile wall itself did not suffer any bending, but rotated as a rigid body as seen in Figure 15. An additional straight line is added in this figure, which extends the measured line of sheet pile wall into the soil. As indicated by this line, we can see that the point of rotation is about a point close to the base of the sheet pile wall.

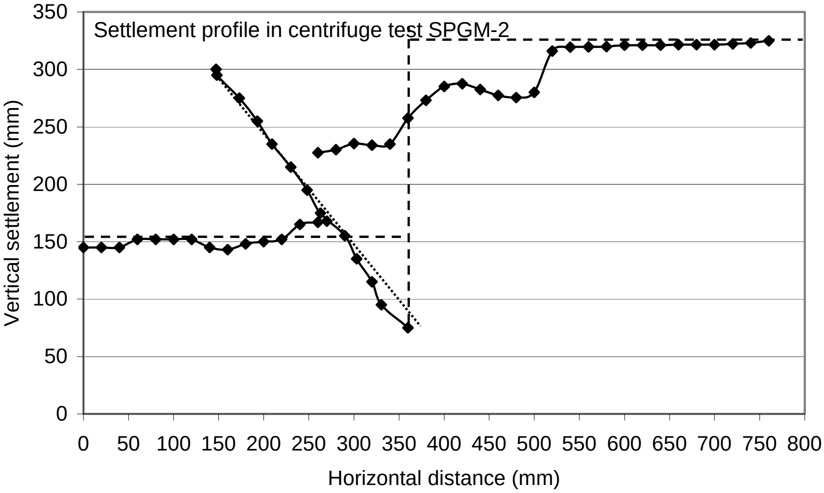


Fig. 15 Measured Settlement and Heave of Clay and the Rotation of the Model Sheet Pile Wall after Centrifuge Test SPGM-2

Implications of the failure mechanism

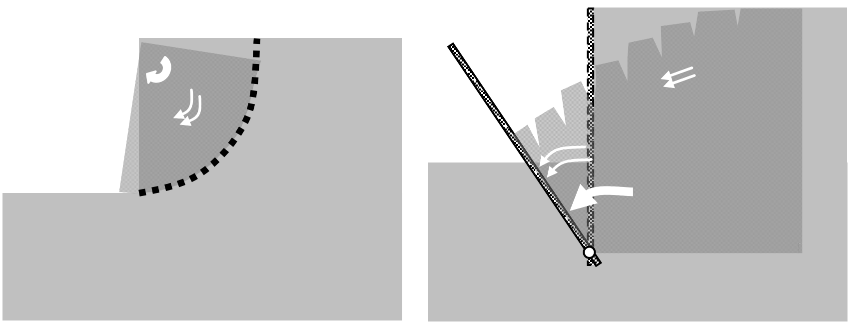
In both the centrifuge tests the predominant failure mechanism was due to rotation of the sheet pile wall about a point close to the base. This mechanism did not change even though the depth of penetration in the second test was significantly lower than in the first. The centrifuge tests revealed this failure mechanism involved excessive settlements and tension cracking of the backfill and significant rotation of the sheet pile walls. The information obtained by the in-flight instrumentation, in-flight digital images and post-test measurements all confirm this failure mechanism. No perceivable lateral translation of the toe of the sheet pile wall in the forward direction was observed in either centrifuge test.

Using Taylor's stability number (Craig, 1987) for a vertical cut in clay the depth to which a tension crack occurs in the clay backfill can be calculated using the equation,

$$H_{crit} = 3.83 \cdot \frac{c_u}{\gamma} \tag{3}$$

Taking the undrained shear strength c_u obtained for the clay from the UCS tests of 30 kPa and the unit weight of the clay as 18.3 kN/m^3, the depth of the tension crack can be determined using Equation 3 to be 6.28m. Using the scaling ratio of 1/N where N = 50 this can be converted to the depth of the tension crack in the model of 125.6mm. Again this corresponds well with the observed tension cracks in the centrifuge test, which extended to a depth of 110mm.

If one imagines a vertical cut in clay, the failure of the vertical cut occurs if the height of the cut increases to that described by Equation 3. It is well established that such a failure would definitely be in the form of a slip failure, which can be determined analytically. The flow of material in this case would be as shown in Figure 16a with the retained earth flowing/rotating in clockwise direction. By placing the sheet pile wall this mechanism is changed as shown in Figure 16b.



a) Soil movement following failure of a vertical cut

b) Soil movement following rotation of a sheet pile wall

Fig. 16 Failure Mechanisms and Soil Movements

Once the sheet pile wall suffers rotation, the soil would now flow/rotate in an anti-clockwise direction. The centrifuge tests presented in this paper confirm that this is both a plausible and a preferred failure mechanism, at least for the depths of penetration of the sheet pile wall studied here. Physical testing using the centrifuge would enable us to observe such mechanisms. The observed failure mechanism of the sheet pile wall is similar to that presented in Figure 1 commonly used by many researchers, for example Broms (1995). The centrifuge test results show that the predominant failure mechanism is via rotation of the wall for sheet pile walls retaining such clays. Also the results indicate that the point of rotation is very close to and above the base of the sheet pile wall.

Implications to cantilever sheet pile walls in the field

The centrifuge test data from this series of experiments can also be interpreted in another way that is more appropriate to Geotechnical Practice. As mentioned earlier the model sheet pile wall represents prototype walls of increasing height as the g level is increased. As the lateral displacement of the wall is known at each g level (for example, see Figures 7 and 13) it is possible to calculate the deflection of the top of the wall. This deflection can be normalised with respect to the height of the backfill to represent overall strain (δ/H). Based on centrifuge test data from SPGM-1, the prototype wall heights are plotted against these overall % strains in Figure 17.

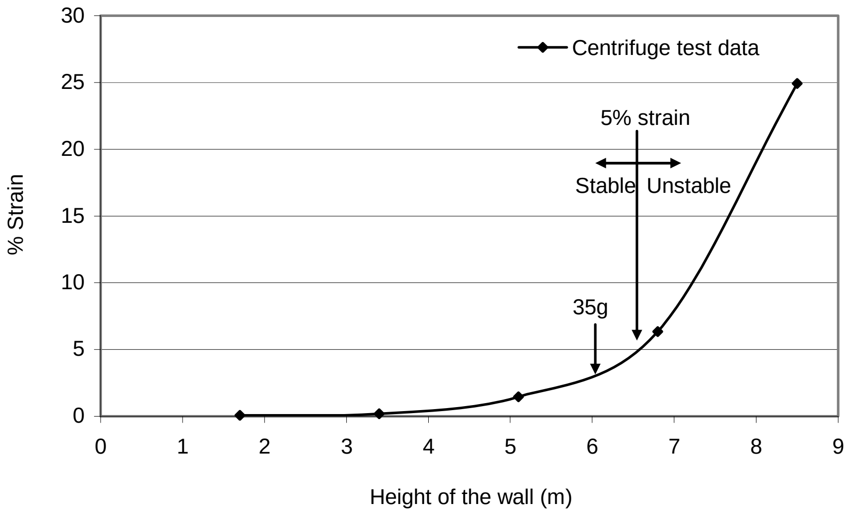


Fig. 17 Stability of Cantilever Sheet Pile Walls with Clay Backfill

In this figure it can be seen that the 5% strain suggested by Azizi (2000) clearly demarcates the stable and unstable regimes for sheet pile walls of different heights in the soft soil considered in this paper. Cantilever sheet pile walls with heights below 6.8m mobilise strains lower than 5%. Further small increase in wall height begets only small increase in % strains. For cantilever sheet pile walls that are above this height the strains that are mobilised are not only large but increase dramatically for small increases in wall heights. This is clearly unstable regime and the stability of the wall may be compromised with any further increase in the wall height.

In the centrifuge test SPGM-1, the model sheet pile wall was stable up to a g level of 35g. Above this g level the % strains have increased resulting in excessive lateral displacement of the sheet pile wall. Also as explained before, the failure was rather sudden and the timing of the failure was dependent on the formation of tension cracks in the backfill. In Figure 17 the 35g was marked by which stage the % strains were below 5%. As the g level was increased beyond 35g the sheet pile walls became unstable and suffered large deformations.

In Geotechnical Practice the above result may be significant in establishing the stability of cantilever sheet pile walls. Also in the instances where additional surcharge loading is expected on the backfill of an existing cantilever sheet pile wall, similar centrifuge experiments can be carried out to establish the safety and stability of the walls. The centrifuge test data will not only establish the stability of such walls, but will yield useful information regarding the deformations that may be suffered by the walls under additional surcharge loading. Serviceability criterion can then be applied to determine whether such deformations are acceptable. While these are qualitative results for the soft clay backfills obtained based on a limited number of centrifuge tests, it is possible to embark upon a more exhaustive series of centrifuge tests to establish the behaviour and stability of retaining walls with more complex and realistic soil profiles and situations. For example a layered backfill or a changing water table can be easily modelled in a centrifuge test series.

Conclusions

In this paper cantilever sheet pile walls that are retaining cohesive backfill are considered. The conventional understanding is outlined with the anticipated net earth pressure distributions that are used routinely in Geotechnical Practice. These are based on the assumption that the sheet pile wall would rotate about a point above the base of the wall. The centrifuge experiments confirmed the failure mechanism employed in Practice and further provided most important information pertaining to deformations that would be required to mobilise full passive resistance.

Two centrifuge tests were conducted on sheet pile walls with varying depths of penetration. The tests were carried out quickly to ensure undrained behaviour of the soil. Also the soil used was soft with undrained shear strength c_u of 30 kPa. In both centrifuge tests the failure mechanism that was observed was similar. The sheet pile walls suffered severe rotation about a point just above the base subjecting the backfill soil to active earth pressures and the soil in front of the toe to passive earth pressures. As the sheet pile wall rotated outwards, excessive tension cracking was observed in the backfill. Passive wedges developed in the toe region of the wall. Also horizontal tension crack was observed on the passive side.

In-flight measurement of the wall deflections and soil settlements, digital images captured in-flight, post-test measurements all confirm the failure mechanism of the sheet pile wall. Further the depth of the tension cracks observed matches satisfactorily with the theoretical estimates. A comparison is made between soil failing in an unsupported vertical cut by forming the traditional slip circle and the soil failing behind a rotating sheet pile wall. The soil flow is seen to be reversed in the presence of the sheet pile wall. Failure mechanisms observed in the centrifuge tests corroborate this reversal of soil flow.

Finally centrifuge modelling is a powerful tool available to the modern day geotechnical engineer. Even though the problem of sheet pile wall considered in this paper is well researched, it establishes the strength of the centrifuge modelling technique in capturing the failure mechanism and the deformation patterns. Even in simple cases considerable insight can be gained by using this technique. Of course it would be a valid tool to investigate more complex problems, for example sheet piles driven into layered soils. The knowledge gained from centrifuge tests can help the geotechnical engineer to come up with innovative economic solutions that are ‘proved’ to be both safe and workable, well before the construction takes place in the field.

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