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Full Length Research Paper

An instrumented reinforced earth wall

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Reinforced earth wall system has become very popular because of its aesthetic value and ease of construction. In order to optimize the design, it is important to understand its behavior during and after construction. This can be achieved either through field instrumentation or numerical simulation. In this paper, field instrumentation work carried out on a reinforced earth wall is described with the objective of capturing the essential behaviour of the wall during and after construction. Two different wall conditions were considered, that is, in the first case, the wall is allowed to move laterally and in the other case, the wall is not flexible in transverse direction. Based on the observations made, it can be concluded that the locus of maximum tension is a vertical line offset approximately 0.35 H away from the facing. For lateral pressure, it is seen that the curve for the flexible facing follows similar trend as the curve for the rigid facing except that the horizontal pressure at the flexible facing is generally slightly lower. Lastly, the connection ratio at rigid facing is generally higher than the connection ratio at the flexible facing.

Key words: Reinforced earth, instrumentation, anchor blocks, lateral movement.

INTRODUCTION

When the reinforced earth system was first introduced by Henri Vidal, there was a large amount of research being poured into it in an attempt to understand its behavior and thereby able to establish a rational basis of design. The research culminated in the publication of the French code of practice for reinforced earth in 1980 called “Reinforced Earth Structures – Recommendations and rules of the art”. For the first time, a rational basis of design using the coherent gravity method is explained in the code, the French code is followed by the design and construction guideline published by the Federal Highway Administration (FHA) in USA (Christopher et al., 1989). The FHA guideline is followed by the National Concrete Masonry Association design manual (Simac et al., 1993) and the British Code (BS 8006:1995). Each successive code and design manual further refines the earlier codes. Despite the successive refinements in the design methodology or approach, the fundamental design philosophy remains the same, that is, it is based on the limit equilibrium method. The basic design assumptions of all these codes and manuals are that the reinforced soil structure is sitting on a firm ground or piled foundation and that there is little yielding of the lateral boundary. In other words, the present design methodology is unable to take into account the effects of yielding at the base and as well as at the facing of the wall. Hence, if the wall is sitting on a compressible founding soil layer, the present design method is unable to capture the stress changes in the reinforcing elements of the wall as a result of the yielding base boundary. Likewise, if the facing of the wall is allowed to move laterally, the present design method is again unable to capture the changes in the tensile stresses developed in the reinforcing elements due to the lateral yielding.

Therefore, the main objective of the study is to investigate the effect of lateral yielding of the facing on the behaviour of a full scale reinforced earth. A patented anchored reinforced earth called Nehemiah Wall is chosen and the instrumentation and monitoring works are carried at two sections of the wall where, at one of them, a polystyrene foam is inserted at the backface of the wall panel to allow for lateral deformation to take place.

NEHEMIAH WALL

The Nehemiah anchored earth wall was first developed and introduced in Malaysia in 1993. The system has been used all over Malaysia and is now being implemented in countries like Singapore,

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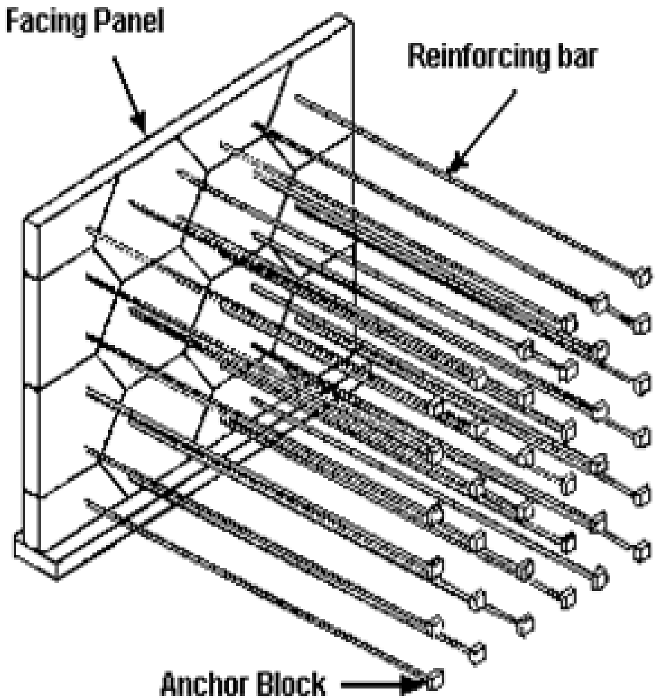

Figure 1. Schematic representation of an anchored reinforced earth wall.

India, Sri Lanka and Bangladesh. Majority of the applications of Nehemiah wall are found in infra-structural projects like highway interchanges, bridge abutments and highways going through mountainous terrain like KL-Karak Highway and Second East-west Highway. A detail description of the various applications of Nehemiah wall can be found in Lee (2000, 2001) and Ali (1990, 2003). The case history of the design, construction and performance of a Nehemiah wall for a bridge over rail project in Malaysia is reported by Lee and Oh (1997). Lee and Nilaweera (2002) reported the construction of a 20.5 m high Nehemiah wall in the Cameron Highlands; this is the highest Nehemiah wall built so far. A schematic representation of the Nehemiah wall is shown in Figure 1. The system is similar to the Vidal system except that the reinforcements consist of round steel bars with precast anchor blocks attached at the free ends of each of the reinforcing bars. One of the advantages of this system is that it can be applied in soils which have relatively high fine-content such as residual soil (Bujang et al., 2008; Ali, 1993; Ali et al., 1992; Anderson et al., 1987; Normaniza et al., 2008). The presence of anchor blocks would help to mobilize additional lateral resistance through bearing.

CONSTRUCTION OF FULL SCALE WALL

The full scale wall was designed according to BS 8006:1995. It was constructed as part of a development project and to support an access road. In view of the large height of the wall, it was designed into two tiers with the upper tier being offset from the lower tier by a distance of 1.5 m. The subsoil consists of an upper stratum of firm to stiff sandy clayey silt. Generally, the shear strength of the stratum increases with depth. The sandy clayey silt stratum is underlain by highly decomposed shale.

River sand was used as backfill material. Resistivity test was carried out and found to range from 12500 - 14440 Ω cm, which was considered as non-corrosive and suitable for use as backfill. The pH value of the sand was found to be 5.5. The resistivity and pH were measured to ensure that the backfill material is not corrosive. The shear strength parameters are obtained from shear box test. The angle of friction was found to be 42 degrees at peak stress while at residue stress the angle was 33 degrees. Conservatively, an angle of friction of 36 degree at peak stress was assumed in the design. The cohesion is taken to be zero.

INSTRUMENTATION

Two sections were selected for instrumentation. At one of the sections, a polystyrene foam was inserted at the backface of the wall panel to allow for lateral deformation to take place. At the other location there was no polystyrene foam insertion, which means that the facing is less flexible in the transverse direction. The instrumented sections are shown in Figures 2 and 3. Basically, the instrumentation consists of inclinometer, rod settlement gauge and load measurement along the reinforcing bars at selected levels.

RESULTS AND DISCUSSION

For rigid facing

As mentioned earlier, the tensile forces along the reinforcing bars were measured by resistance wire strain gauges. Figure 4 shows the distribution of the tensile forces along the reinforcing bars at their respective levels. It was seen that the maximum tensions occur at distance away from the facing. The locus of maximum tension is more or less a vertical line at 0.35H (where H is the overall height of the wall) offset from the backface of the wall facing. In a tie back system; the tensile force in the reinforcing bar is constant throughout the length of the bar. Hence, the variation of the tensile force along the reinforcing bar confirms that Nehemiah wall is indeed a reinforced soil system rather than a tie back system.

The variation of the horizontal pressure with the normalized depth below the crest of wall was plotted and shown in Figure 5. The K_a line is drawn in the same plot for comparison. Assuming angle of friction of 36°, K_a = 0.26. Whereas, by assuming an angle of friction of 30°, K_a = 0.33. The K_a = 0.33 line seems to be a more realistic representation of the measured results. Based on K_a =0.33 line, it is seen that for normalized depth less than 0.4, the horizontal pressure is above the K_a line. The explanation for the higher horizontal pressures at shallow depths is that they are induced by the locked in stresses due to compaction. For normalized depth greater than 0.7, the horizontal pressure is below the K_a line. Between the normalize depths 0.4 and 0.7, the horizontal pressures follow closely the K_a line. The approximate parabolic shape of the pressure distribution indicates that perhaps the wall at the toe deforms significantly more, thereby resulting in lower horizontal pressure.

The variation of coefficient of lateral pressure with depth is plotted as shown in Figure 6. The K_a=0.26 and K_a = 0.33 are plotted. The K_a = 0.33 line selected for comparison. The coefficient of lateral pressure plot tells a

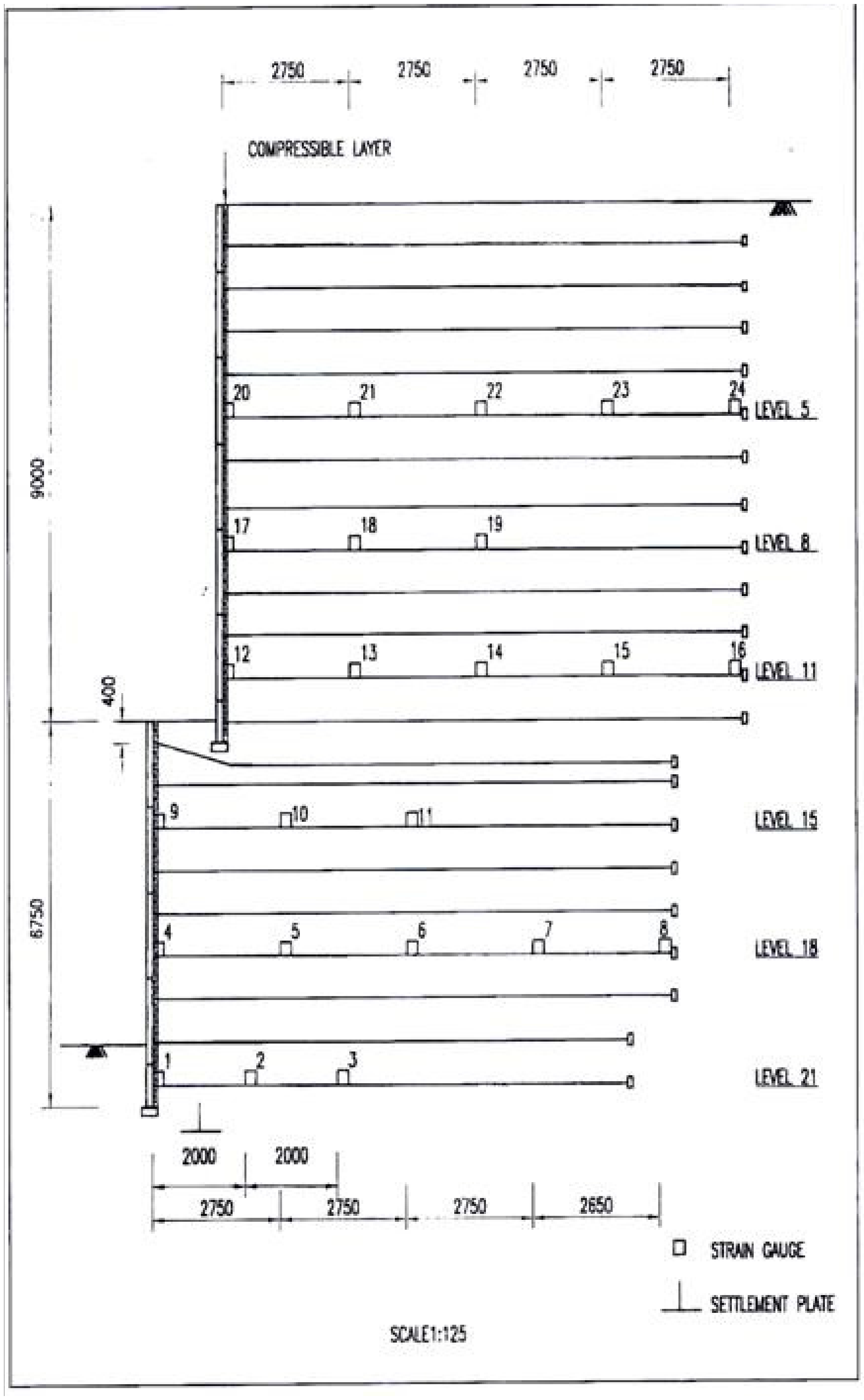

Figure 2. Instrumented section of the wall (rigid facing).

similar story as the horizontal pressure plot.

For shallow depths, that is, d_n less than 0.4, the coefficient of lateral pressure far exceeds even the K_a value. Whereas, for deeper depth where d_n is greater than 0.8, the k value is far below the K_a value. For d_n between 0.4 and 0.8 the k value is pretty well represented by K_a value. Connection ratio is an important parameter because the tensile strength of the reinforcing bar is generally the weakest at the connection. The connection ratio helps to determine whether the critical section is located at the connection or otherwise. The connection ratio is plotted against the normalized depth and shown in Figure 7. It is seen that the C_r fluctuates between 0.4 and 0.7 at shallow depths. When the overburden is larger than 0.7 H,

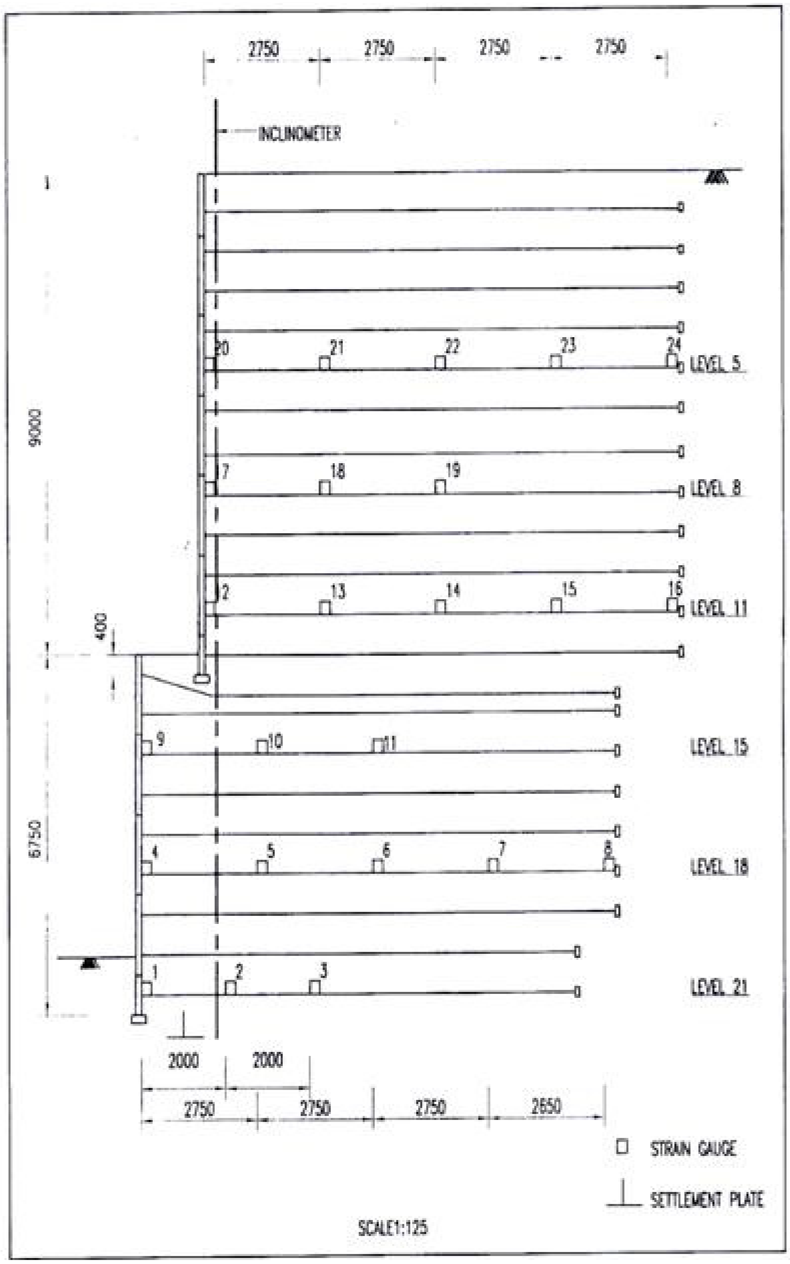

Figure 3. Instrumented of the wall (flexible facing).

the Cr decreases with depth till it reaches zero at the toe of the wall. This is a significant departure from the behavior of Vidal (reinforced earth) wall where the connection ratio increases with depth till it is 1.0. A possible explanation for this phenomenon is that the wall facing at the lower part of the wall deforms significantly more than those measured in Vidal walls.

For flexible facing

This is another section of the wall chosen for instrumentation. The geometrical dimensions and design of the wall are similar to the rigid wall facing. The only difference is that at this section a layer of compressible geoinclusion made of polystyrene material is inserted at the backface of the wall to allow lateral deformation to take place. The purpose of introducing the compressible layer is to study the effect of yielding lateral boundary condition. The tensile forces were measured by the strain gauges. The tensile force distribution along the rein-forcing bars at each of the instrumented level is shown in Figure 3. For the purpose of comparison, the tensile force distribution is superimposed on the tensile force distribution of the

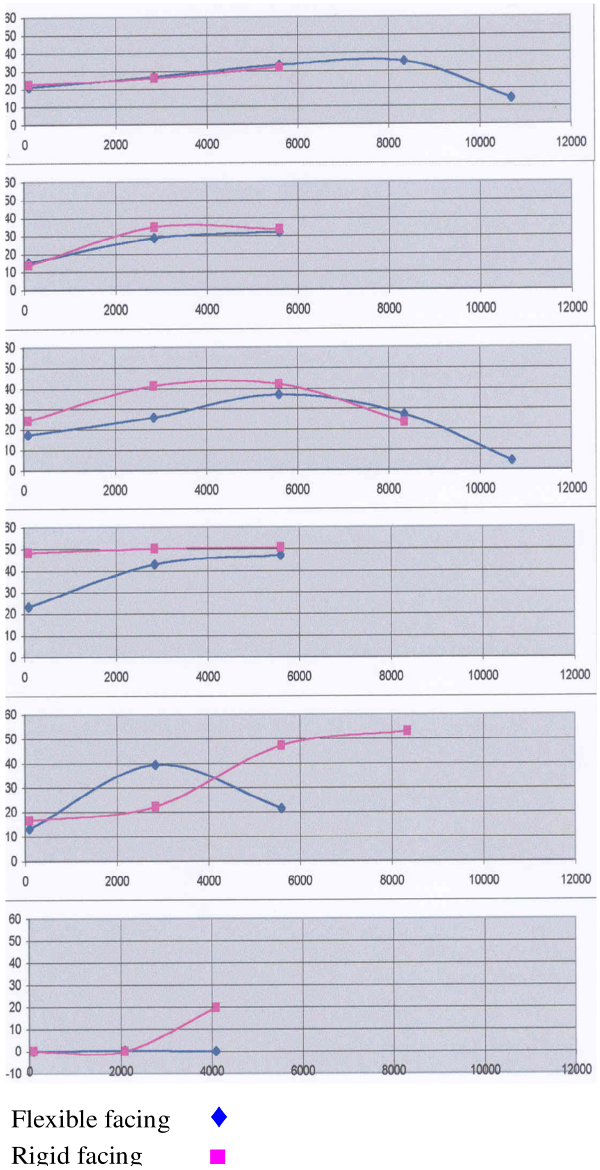

Figure 4. Tensile force distribution along the reinforcing bars at both sections.

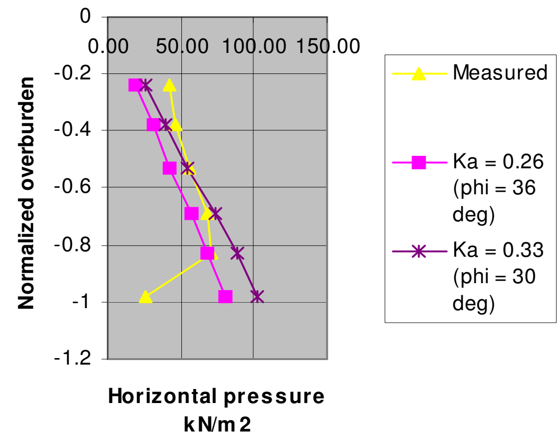

Figure 5. Variation of horizontal pressure with overburden.

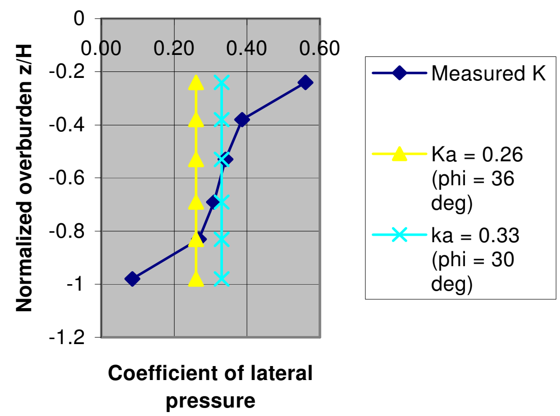

Figure 6. Variation of coefficient of lateral earth pressure with depth of overburden.

rigid facing. It is seen that the trend of the locus of maximum tension in both sections is similar; that is, the locus is a vertical line offset approximately 0.35 H away from the facing.

The horizontal pressure is plotted against normalized depth (Figure 8). The horizontal pressure increases with depth until d_n reaches 0.7 whereby the horizontal pressure starts to decrease with greater depth. It is seen that the curve for the flexible facing follows similar trend as the curve for the rigid facing except that the horizontal pressure at the flexible facing is generally slightly lower. The variation of coefficient of lateral pressure with overburden for the flexible facing is shown in Figure 9. It is seen that at shallow depths, the coefficient is much higher than K_a. Then as the depth increases, the coefficient follows more or less the K_a line. Below an overburden depth of 0.7, the coefficient fall below the K_a value.

The connection ratio of the flexible facing is plotted against the normalized depth. The maximum value of C_r is 0.6 and maintains more or less a constant value of approximately 0.48 for depth between 0.38 and 0.69. Then the C_r value drops rapidly to zero toward the base of the wall. The connection ratios at the rigid facing are also plotted in Figure 10. The connection ratio at rigid facing is generally higher than the connection ratio at the flexible facing.

Conclusions

A high anchored reinforced earth wall was fully instrumented to monitor the behaviour during and after construction.

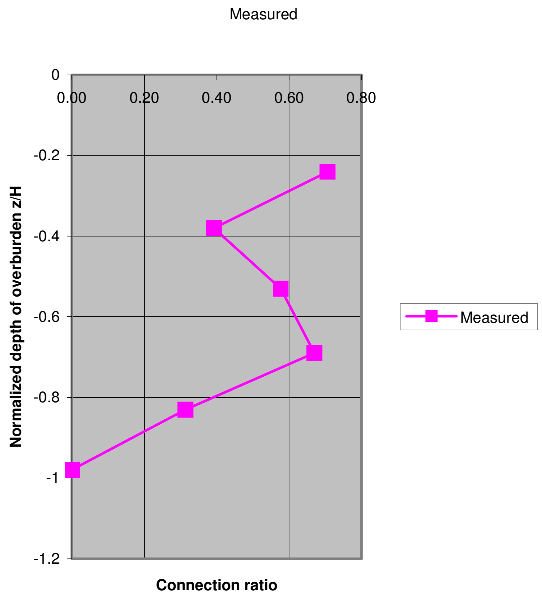

Figure 7. Variation of connection ratio with depth of overburden.

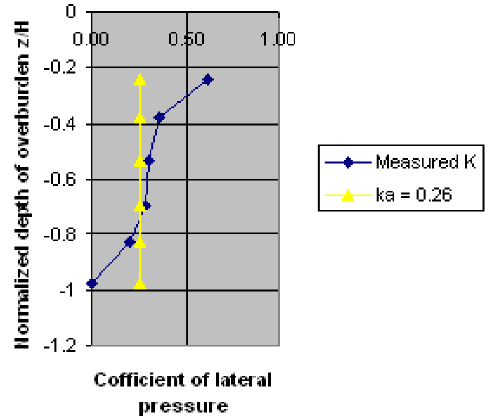

Figure 9. Variation of coefficient of lateral pressure with depth of overburden.

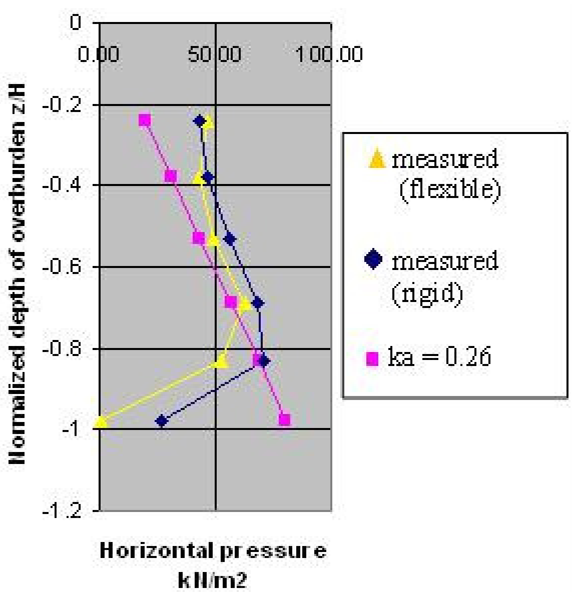

Figure 8. Variation of horizontal pressure with depth of overburden.

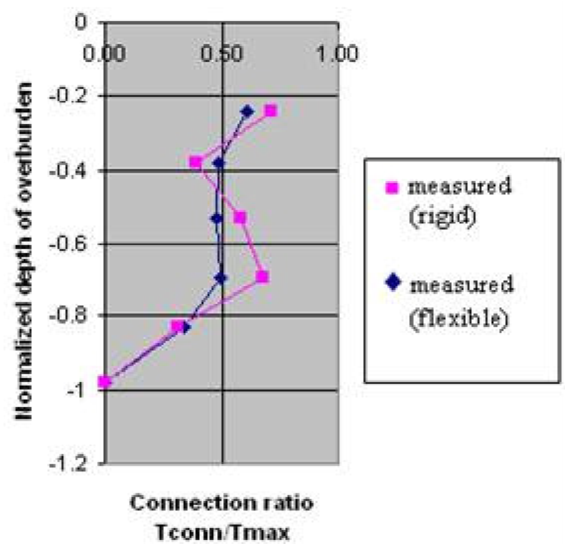

Figure 10. Variation of connection ratio with depth of overburden.

The focus is on the difference in behaviour between rigid and flexible wall. One of the walls was made less rigid by inserting compressible geoinclusion made of polystyrene material at the backface of the wall to allow lateral deformation to take place. The parameters measured include stresses along the reinforcement, lateral pressure in the backfill, and lateral deflection of the wall.

Based on the observations it is concluded that, the trend of the locus of maximum tension in both sections is similar; that is, the locus is a vertical line offset approximately 0.35 H away from the facing. For lateral pressure, it is seen that the curve for the flexible facing follows similar trend as the curve for the rigid facing except that the horizontal pressure at the flexible facing is generally slightly lower. Lastly, the connection ratio at rigid facing is generally higher than the connection ratio at the flexible facing.

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