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2019

Rahardjo, H., Gofar, N., Satyanaga, A., Leong, E. C., Wang, C. L. & Wong, J. L. H. (2019). Effect of rainfall infiltration on deformation of Geobarrier Wall. *Geotechnical and Geological Engineering*, 37(3), 1383-1399. <https://dx.doi.org/10.1007/s10706-018-0693-6>

<https://hdl.handle.net/10356/151491>

<https://doi.org/10.1007/s10706-018-0693-6>

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EFFECT OF RAINFALL INFILTRATION ON DEFORMATION OF GEOBARRIER WALL

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Abstract

GeoBarrier system (GBS) is a system that combines geobag wall with capillary barrier concept as stabilization method against rainfall-induced failure, and vegetation as green cover. This paper presents deformation behaviour of a GBS wall constructed as pilot study when subjected to rainfall infiltration. The GBS wall was 4 m high with 70° slope angle. The wall was subjected to uniform surcharge of 10 kN/m² and rainfall of 22 mm/h for 8 h. Recycled materials were used as components of the capillary barrier. Coupled deformation-seepage finite element analyses were carried out to evaluate the stress and pore-water pressure variations within and behind the GBS wall during and after rainfall. Limit equilibrium analyses were performed at various stages during and after rainfall to assess the near vertical wall stability. The results of the coupled deformation seepage analysis showed that the reinforced zone in the GBS remained unsaturated during rainfall, indicating that the GBS wall performed well in

minimizing the rainwater infiltration. However, the GBS wall face deformation pattern altered during and after rainfall, indicating that the rainfall infiltration increased the lateral stress on the top part of the wall. The variation of lateral stress with depth agrees with field measurements from earth pressure cells.

Keywords: GeoBarrier System, rainfall infiltration, pore-water pressure, deformation, recycled material.

1. INTRODUCTION

Retaining walls are important part of urban development related to the creation of new spaces for urban development where land space is very limited. Conventional concrete retaining walls are usually associated with high cost, poor aesthetic and lengthy construction time. Drainage provisions are required to maintain the wall's stability when exposed to rainfall. Reinforced soil walls have been used as an alternative to the concrete walls. However, the construction of reinforced soil wall requires the use of granular, non-cohesive soil as reinforced backfill materials to ensure adequate drainage. Koerner and Koerner (2013) compiled a database of failed reinforced soil structures and they concluded that most of the failures were related to the use of low quality fill material and poor drainage system. Detailed study on the failure of segmented reinforced wall reported by Yoo and Jung (2006) also showed that although the failure was triggered by rainfall infiltration, inappropriate design and the low-quality backfill were the main factors that cause the failure. Thus, it is important to protect the reinforced soil fill material from rainfall infiltration. Rahardjo et al. (2015) proposed a GeoBarrier system (GBS) that serves two functions: as a retaining wall and as a cover to minimize rainwater infiltration to maintain the stability of the retained slope subjected to frequent rainfalls.

The GBS utilizes geobag type of wall (Matsuoka and Liu, 2006) to improve the near vertical slope's stability and to utilize capillary barrier principles to reduce rainwater

infiltration into the reinforced soil (Rahardjo et al., 2015). A geobag is a type of gabion made from geosynthetic material and filled with granular soils. These bags are interlayered with reinforcing elements fixed in-between the bags. The reinforcing element increases the stability of the GBS wall through interface friction between the reinforcing element itself (made of geogrids) and the reinforced soil (made of compacted in-situ soil). The system is enhanced with green cover at the wall face.

The GBS is a three-layer cover system designed as a vegetative layer over a two-layer capillary barrier system. The capillary barrier utilizes the contrast in hydraulic properties of fine- and coarse-grained layers in unsaturated conditions to limit water percolation into soil and therefore the loss of matric suction (Stormont and Anderson, 1999; Khire et al., 2000; Tami et al, 2004, Harnas et al, 2014). In the design of the GBS wall, the geobag is filled with fine-grained materials and laid over a layer of coarse-grained material to form a capillary barrier system on the wall face. The vegetative cover is made of deep-rooted grass or shrubs planted in bags of planting soil or approved soil mixture (ASM). The bags of ASM are laid over the bags of fine-grained material to facilitate the growth of vegetation. Rainwater that infiltrates and seeps through the geobags were channelled into a surface drain or sump at the toe.

By utilizing the capillary barrier principle, the GBS can minimize rainfall infiltration into the reinforced soil through capillary barrier effect at the interface between the fine- and coarse-grained materials (Rahardjo et al. 2015). The capillary barrier system (CBS) has been shown to perform well in minimizing rainfall infiltration into gentle slope (Rahardjo et al., 2007; 2012a). Field studies (Stormont, 1996; Rahardjo et al, 2006; 2011) have shown the effectiveness of the capillary barrier system does not only depend on the contrast in the hydraulic properties of the fine- and coarse-grained materials and the storage capacity of the fine layer, but also on the meteorological conditions of the site. The performance of the two-

layer capillary barrier for a tropical climate with prolonged rainfall still needs to be evaluated. Some researchers proposed three-layer capillary barrier (Zhan et al, 2014; Ng et al, 2016) or dual capillary barrier (Harnas et al, 2014).

The GBS employs recycled materials as part of efforts to support a green system that reduces the carbon footprint. Every year about two million tons of waste from construction and demolition in Singapore are recycled and available for reuse (Building Construction Authority Singapore, 2008). The Building and Construction Authority and National Environmental Agency in Singapore are attempting to work closely with industries to promote the use of recycled materials in building and construction projects to achieve environmental sustainability and increase environmental awareness. In addition, the use of recycled concrete aggregates (RCA) and reclaimed asphalt pavements (RAP) are less expensive than the use of natural aggregates, thereby reducing construction costs. For these reasons, RCA and RAP were adopted as the fine- and coarse-grained materials in the design of GBS. Recycled materials have been used worldwide in various geotechnical engineering applications (Viera and Pereira, 2015; Cardoso et al., 2016). The unsaturated properties of recycled materials were studied by Rahardjo et al (2013). In GBS, fine RCA or fine RAP was wrapped in a geobag and stacked on top of each other on the wall face to form a retaining wall. Coarse RCA or coarse RAP was placed behind the geobags to limit water infiltration into the reinforced soil.

Numerical study was performed to analyse the deformation behaviour of the GBS wall when subjected to rainfall infiltration. The dimensions and material properties used in the analysis were based on a field study carried out in Singapore. The walls were instrumented and monitored for one year. Coupled deformation–seepage analyses were carried out using finite element software SIGMA/W (Geoslope International, 2012a), which is capable of undertaking seepage analysis coupled with stress/deformation analysis, to observe the transient pore-water

pressure and to evaluate the deformation of the GBS during construction, end of construction, under surcharge load as well as during and after rainfall. Coupled deformation–seepage analysis has been used by researchers (e.g. Wu et al., 2016 and Qi and Vanapalli, 2015) to analyse infiltration **problems** and to study the effect of infiltration on slope stability. The same approach was also used by Rahardjo et al. (2015) for preliminary study of GBS wall. Based on the comparison between the physical model and numerical analyses of CBS, Tami et al. (2004) showed that the numerical results of the model using wetting SWCC and wetting permeability function matched well with the measured data from the physical model during the precipitation process. Thus, hysteresis behaviour of hydraulic properties of soil was accounted for in this numerical modelling in order to provide realistic results. Limit equilibrium method in SLOPE/W (Geoslope International, 2012b) was adopted to evaluate the global and local stability of the wall as well as the tensile force resisted by the geogrids reinforcement at the respective stages. The results in terms of transient pore-water pressure distributions and transient earth pressure were verified using field data.

2. FIELD STUDY

Three GBS walls were constructed on residual soil from Bukit Timah Granite in Singapore to study the response of the wall to flux boundary conditions especially rainfall infiltration (Figure 1). The residual soils cover almost the entire Singapore, which are classified as residual soils from Bukit Timah Granite, Jurong Formation and Old Alluvium. The residual soil of Bukit Timah Granite was formed by weathering of the granite at the central part of Singapore Island (Rahardjo et al., 2012b). The weathering profile of the residual soil of Bukit Timah Granite is more uniform compared to the residual soil of the sedimentary Jurong Formation (Rahardjo et al., 2012b). The residual soil varies from silty sand, sandy silt, clayey silt to silty clay with plasticity within the range for kaolinite in the plasticity chart. The soil's

consistency ranges from medium stiff to very stiff because the degree of weathering decreases with depth (Rahardjo et al., 2011).

Figure 1 Field study of GeoBarrier System (GBS) wall in Singapore

The study area was instrumented with a rain-gauge and two Cassagrande-type piezometers to monitor rainfall and groundwater table, respectively. Each GBS wall was instrumented with four pairs of tensiometer and soil moisture sensor for the measurement of negative pore-water pressures and soil moisture changes due to rainwater infiltration. In addition, earth pressure cells were installed behind the GBS slopes to monitor the transient total vertical and lateral stresses in response to rainwater infiltration. All instruments (tensiometers, soil moisture sensors, earth pressure cells, piezometers, rain-gauge) were connected to a data acquisition system (DAS) and sent to a website for remote real-time monitoring. The responses of GBS walls were monitored for a period of 1-year from July 1st 2016 to June 30th 2017.

The GBS walls were 4 m high sloping at 70° to the horizontal (Figure 2). Geobags (0.5 m × 0.5 m × 0.5 m) filled with fine RCA or fine RAP and bags of approved soil mix (ASM) of

0.6 m $\times$ 0.5 m $\times$ 1.5 m serves as facing panel of the GBS wall. The GBS consists of eight layers of geobags with a 2.8 m long geogrids reinforcement. The geogrids reinforcements were secured in between the geobags of fine grained material and ASM (Figure 2). The in-situ soil was compacted to 90% relative density and used as reinforced soil fill. A 0.3-m thick of coarse-grained material (coarse RCA or coarse RAP) was laid on the reinforced soil before the placement of bags of fine RCA or fine RAP. In this pilot study, the GBS walls were designed with three different combinations of materials as the coarse-grained layer (RCA or RAP), fine-grained layer (RCA or RAP) and the planting soil (ASM or Mixture of ASM with RCA). However, this paper only focuses on the GBS wall with fine RAP and coarse RAP.

The global and local stability of the GBS wall were assessed prior to construction using a simplified method (Building Construction Authority Singapore, 2009). The results showed that the wall was stable with FoS against sliding of 1.75, and global slope stability of 2.04. Overturning is rarely an issue in reinforced soil wall because the structure is flexible (Koerner, 2005). Similarly, bearing capacity failure and excessive settlement are not of concern for reinforced soil wall because the structure is light and flexible, respectively. The schematic diagram of the GBS walls and locations of the instruments are shown in Figure 2.

carried out to evaluate the wall deformation at various stages before, during, and after rainfall. The global and local stability of the wall and the tensile force resisted by the geogrid reinforcement at these respective stages were evaluated using the limit equilibrium method.

The finite element model adopted in this study is a two-dimensional plane strain model.

The soil particles were treated as continuum and they were modelled using elastoplastic constitutive model in SIGMA/W. Two stress state variables were employed in this analysis to describe the unsaturated soil constitutive relationships. The two incremental constitutive equations for an unsaturated soil can be written as follows:

$$d\varepsilon_{ij} = \frac{1+\nu}{E} d\sigma_{ij} - \frac{\nu}{E} d\sigma_{mean} \delta_{ij} - \frac{1}{H} du_w \delta_{ij} \quad (1)$$

$$d\theta_w = \frac{\beta}{3} \varepsilon_v - \frac{1}{R} du_w \quad (2)$$

where ε_{ij} is the components of the strain tensor, ν is the Poisson's ratio, E is the elastic modulus for the soil structure, σ_{mean} is the mean total stress, δ_{ij} is the Kronecker delta, H is the elasticity parameter for the soil structure with respect to a change in matric suction, $\beta = E / H(1-2\nu)$, ε_v is the volumetric strain, and R is the parameter relating the volumetric water content to a change in pore-water pressure. A typical value of Poisson's ratio, ν , of 0.33 was assumed in SIGMA/W analyses. Substituting Eq. (1) into Eq. (2) yields the Eq. (6) relating the change in volumetric water content to change in volume of soil structure:

$$d\theta_w = \beta d\varepsilon_v - \omega du_w \quad (3)$$

$$\text{where } \omega = \frac{1}{R} - \frac{3\beta}{H}.$$

The boundary conditions applied to the finite element model are illustrated in Figure 3. The left and right boundaries were set at 10 m from the crest and toe of the sloping wall, while the bottom boundary was set at 8 m from the ground surface at the toe of the slope. The mesh consisted of 3593 rectangular and triangular elements. For deformation analysis, zero vertical displacements were assigned at both vertical boundaries and the bottom of the model. For seepage analysis, no flow boundaries were simulated by assigning a nodal flux Q equal to zero at the bottom and sides of the slope model above ground water level (GWL). A constant total head h_w was applied on each side boundary below GWL. Rainfall was applied to the ground surface as flux boundary q . Ponding was not allowed to occur on the ground surface. This means that the pore water pressure on the ground surface will not be allowed to be higher than 0 kPa. This condition simulated the actual condition whereby the excess rainwater on the slope would become surface flow or runoff. The construction sequence adopted in the numerical analysis started with an in-situ condition and placement of eight lifts of reinforced layers and geobags (Figure 3), followed by the application of surcharge load (10 kPa) on the crest and rainfall of 22 mm/h for 8 hours. The initial pore-water pressure within the residual soil behind GBS wall was set based on depth of ground water table from the ground surface. The initial pore-water pressure within the GBS was set as constant matric suction of 35 kPa.

Figure 3 GBS wall for numerical analysis using coupled deformation-seepage analysis in SIGMA/W and slope stability analysis in SLOPE/W

3.2 Material properties

The GBS slope analysed in this paper was constructed of compacted residual soil as reinforced soil, fine RAP and coarse RAP as the capillary barrier components and ASM for green cover. Based on the grain size distribution and consistency, the residual soil can be classified based on Unified Soil Classification System, USCS (ASTM D2487) as highly plastic clay (CH), hence sensitive to change in water content. Compacted residual soil was used as backfill behind the GBS wall. The soil was compacted to achieve field density of 2.0 Mg/m^3 (90% maximum dry density). The hydraulic properties of the residual soil and the compacted residual soil were measured in the laboratory following relevant ASTM standards. The drying and wetting SWCC and the permeability function derived from SWCC wetting curve of the residual soil and the compacted residual soil derived using statistical method (Fredlund et al,

1994) are shown in Figure 4. Compaction process resulted in lower soil's porosity, therefore the saturated volumetric water content and the coefficient of saturated permeability decreased.

Figure 4 Drying and wetting SWCC and Permeability function of residual soil and compacted soil

Previous studies indicated that there are several controlling parameters that must be considered in the selection of GBS materials. Based on their study, Smersud and Selker (2001) concluded that the ratio of particle size (defined as d_{50}) of the coarse- and fine-grained material must be greater than 5 to be selected as GBS materials. Rahardjo et al. (2007) suggested that the ratio between the water-entry value (ψ_w) of the fine-grained layers should be one order of magnitude higher than that of the coarse-grained layers (ψ_w -ratio) to create significant barrier effects between fine- and coarse-grained layers. Rahardjo et al. (2007) also suggested that the ψ_w of coarse grained material is preferably less than 1 kPa and the saturated coefficient of permeability (k_s) of the fine-grained layer should be higher than 10^{-5} m/s to allow water to flow out from the fine-grained layer by lateral diversion.

Grain size distribution and hydraulic properties as well as the deformation properties of ASM, fine RAP and coarse RAP were evaluated in the laboratory following relevant ASTM standards. Based on grain size distribution and consistency, the ASM could be classified based

on USCS (ASTM D2487) as silt with low plasticity (ML) while fine RAP and coarse RAP are classified as poorly graded sand (SP) and poorly graded gravel (GP) respectively. The grain size distribution of the ASM, fine RAP and coarse RAP are shown in Figure 5. The grain size distributions indicated that the d_{50} of ASM, fine RAP and coarse RAP are 0.22mm, 2.2mm, and 31mm, respectively. Therefore, the ratio of d_{50} of coarse RAP to d_{50} of fine RAP was 14 greater than 5, and the ratio of d_{50} of fine RAP to d_{50} of ASM was 10, also greater than 5. Thus, based on **grain-size distribution** criterion, the combination of fine RAP and coarse RAP is suitable as capillary barrier. In addition, the capillary barrier effect may also exist between ASM and fine RAP.

Figure 5 Grain size distribution of ASM, fine RAP and coarse RAP

The drying and wetting soil-water characteristic curves of ASM, fine RAP and coarse RAP are shown in Figure 6. The water-entry values (ψ_w) of ASM, fine RAP, and coarse RAP are 80kPa, 5kPa and 0.1kPa, respectively. Thus, the ψ_w -ratio between the fine and coarse RAP was 50 while ψ_w -ratio between the ASM and fine RAP was 16. Both ψ_w -ratio was greater than

10, thus both combinations, i.e. ASM and fine RAP as well as fine RAP and coarse RAP, can make a good capillary barrier. The ψ_w of coarse RAP is less than 1 kPa, hence; suitable as coarse grained component of capillary barrier. The ψ_w of fine RAP is higher than 1 kPa, thus based on this criterion, the fine RAP is not suitable as coarse grained component of capillary barrier when combined with ASM. However, the k_s of ASM and fine RAP are 1×10^{-5} m/s and 4×10^{-4} m/s, respectively, which meet the criterion as fine-grained component of the capillary barrier. The saturated coefficient of permeability, k_s equal to or higher than 1×10^{-5} m/s ensures that the water can flow effectively through this soil. In addition, Figure 6 indicates that the wetting and drying curves of ASM display highly hysteretic behaviour, therefore has higher water retention capability.

Figure 6 Drying and wetting SWCCs and permeability functions of ASM, fine RAP and coarse RAP

The modulus and shear strength properties of the residual soil as well as all materials used for the construction of GBS wall including gravel for the construction of sump were tested in the laboratory following relevant ASTM standards. Measurement of saturated shear strength of ASM, residual soil and compacted soil were carried out using multistage consolidated

undrained (CU) triaxial tests with pore-water pressure measurement (ASTM D4767-04). Measurement of shear strength of gravel and recycled materials were performed using multistage consolidated drained (CD) triaxial tests (ASTM D7181-11). Multistage triaxial tests were adopted in the shear strength test to minimise the effect of variability of the specimens on the test results (Rahardjo et al., 1995). The testing procedure is time saving and provide maximum information from a limited number of available soils and materials. A constant axial rate of 0.01 mm/min was adopted in the saturated triaxial tests since it was found to be sufficient for the pore-water pressure to dissipate (Head and Epps, 2011).

The preparation of specimen for the unsaturated triaxial test was similar to that for the saturated triaxial test. However, a 5-bar high-flow high-air entry ceramic disk was placed in the pedestal of modified triaxial cell instead of porous stone. A 16.13 mm thick ceramic disk was saturated prior to unsaturated triaxial tests following the procedure described by Bishop and Henkel (1962). The unsaturated triaxial test consists of four stages, saturation, consolidation, matric suction equalization and shearing stages. The axis-translation technique (Hilf, 1956) was used to apply matric suction during the matric suction equalization stage. The matric suction equalization was terminated when there was no more change in water volume. In the shearing stage, the specimen was sheared at a constant axial rate of 0.01 mm/min since it had been used for CD triaxial tests on compacted residual soil (Rahardjo et al., 2004). The deformation properties of residual soils and construction materials for GBS are summarized in Table 1.

Geogrid was used as reinforcing element of GBS and modelled numerically as structural beam element. The initial characteristic strength of the geogrid at 5% strain was 30 kN/m and unit weight was 0.25 kg/m². Each reinforcing layer comprises two sheets of geogrids with a combined thickness (t) of 4 mm and characteristic strength (T_{ult}) of 60 kN/m. The initial

modulus of the geosynthetic (J) was 600 kN/m. The properties of geogrids, i.e. the modulus of elasticity (E) and the moment of inertia (I) of the beam are also given in Table 1.

3.3 Results of Numerical Analysis

Deformation of the wall was evaluated at the end of construction, upon application of surcharge, and at various intervals during and after rainfall stopped. The transient pore-water pressure (PWP) induced by rainfall infiltration was expected to influence the deformation pattern. Global and local stability analyses were carried out at the corresponding intervals to obtain the factors of safety (FoS) and the maximum tensile stress in geogrids.

Table 1. Deformation Properties used in numerical analyses

Soil	Effective Elastic modulus E' (kPa)	Effective cohesion c' (kPa)	Effective friction angle ϕ' ($^\circ$)	Angle indicating the increase in shear strength due to the increase in suction ϕ^p ($^\circ$)	Unit weight γ (Mg/m ³)	Poisson's ratio ν
Residual soil (1 st layer)	6,000	5	28	17	1.8	0.4
Residual soil (2 nd layer)	15,000	12	32	17	1.8	0.4
Compacted Soil	7,000	3	38	14	2.0	0.4
Fine RAP	120,000	0	34	0	2.0	0.33
Coarse RAP	200,000	0	42	0	2.1	0.33
Gravel	200,000	0	42	0	2.1	0.33
Structural Beam	Elastic modulus (E , kPa)			Moment of inertia (I , m ⁴)		
Geogrids	150,000			5.3×10^{-9}		

The PWP profiles in GBS wall are shown in Figures 7a -7d. Figures 7a show the PWP profile in ASM layer (section 1-1 in Figure 3) while Figure 7b and 7c show the PWP profiles in the fine RAP and coarse RAP layers (section 2-2 and section 3-3 respectively in Figure 3). The PWP in coarse RAP were almost constant (only varied between -4.5 and -5 kPa) indicating that there was no breakthrough from fine layer despite high intensity long duration rainfall (22mm/h for 8h). The PWP in fine RAP increased from -15 kPa to -8kPa during rainfall; the layer never reached saturation. This indicated that the water was mostly stored in the ASM

layer before it was discharged due to the high storage capacity and the thickness of the ASM layer (0.6m). Figure 7a shows that the ASM was always in saturated condition (PWP=0). The PWP profile in the compacted residual soil (section 1-1 in Figure 3) is shown in Figure 7d. The PWP within the reinforced soil zone remained the same because the soil was protected by GBS on the crest and the wall face.

Figure 7 Transient pore-water pressure in (a) ASM (b) fine RAP (c) coarse RAP (d) compacted soil

Contours of PWP 2h, 8h and 12 h after the rain started are depicted in Figure 8. During rainfall, the rainwater only flowed through the ASM layer and it was drained out through the sump below the GBS. Figure 8 also indicated that there were no changes in matric suction within the compacted soil in the reinforced zone behind GBS. This shows that the extension of GBS on the crest performed well in minimizing rainwater infiltration into the compacted residual soil in the reinforced zone behind GBS.

Figure 8 Contour of PWP in GBS wall (a) 2 h after rain started (b) 8 h (rain stop) (c) 12 h after rain started

Figure 9 shows the deformation of GBS wall face and surface displacement at the end of construction and upon the application of surcharge loads. The patterns indicated that the

movement of wall face started at the bottom indicating that the surcharge load was transferred effectively through the reinforced soil. The maximum surface displacement after surcharge was 9mm while the maximum wall face deformation was 6.7mm at the bottom part of the wall. The wall deformation pattern changed significantly after rainfall application (at 8h and 12h). The rainfall induced movement of the upper part of the wall (Figure 9a) and swelling on the surface (Figure 9b). When the rainfall stopped, the maximum surface displacement was 2.5mm while the maximum face deformation was 8.5mm at the top part of the wall. At 12h after the start of rainfall, the ground surface at the crest was back to original position. This indicated that the rain water was stored in the fine layer at the surface and in ASM layer before it flows down along the slope face and through the fine and ASM layer. The accumulated water in the upper part of the GBS wall inducing larger movement at the top, and therefore; influenced the deformation pattern of the wall.

Figure 9 (a) Wall face deformation and (b) Surface displacement of GBS wall

The stability of GBS slope was analysed for two conditions, global and local stability. The global stability analysis was defined by slip surface outside the GBS while the local

stability was defined by slip surface inside the GBS itself as shown in Figure 10a and 10 b respectively. The global and local stability analyses were carried out using limit equilibrium with Morgenstern and Price method in SLOPE/W (Geoslope International., 2012b). This method was selected because it satisfies both force and moment equilibriums. The method also allows the definition of critical slip surface within a certain region by definition of entry and exit points. This method was adopted to differentiate the global and local failure planes (inside the GBS wall). The critical slip surfaces for the global and local slope stability analyses are shown in Figure 10a and Figure 10b, respectively. The variations of FoS during rainfall, shown in Figure 10c indicates that the local stability was not affected by the rainfall because the pore-water pressure in reinforced soil was almost constant (Figure 7b). On the other hand, the global stability was affected by rainfall. Delayed failure was expected because the residual soil is highly plastic clay (CH) with low coefficient of permeability, thus water takes time to percolate.

The effect of rainfall on vertical and horizontal pressures at the position of earth pressure cell (EPC), i.e. at depth 1.5m from crest, in terms of normalized pressure is shown in Figure 11. The figure indicates that the effect of rainfall is more pronounced on the horizontal pressure as compared to the effect on vertical pressure. The horizontal pressure increases by 2.6 times of the initial value after 2h of rainfall application while the vertical pressure increased by only 1.4. The pressure increase was due to the change in PWP of the residual soil behind the GBS wall.

(c) Effect of Rainfall on Slope Stability

Figure 10 (a) Global and (b) local failure plane in GBS wall (c) Effect of rainfall infiltration on the global and local FoS of slope

Figure 11 Effect of rainfall on normalized vertical and horizontal pressure with respect to initial pressure

The increase in the horizontal pressure did not translate to the stress mobilized in the geogrids because this pressure was derived from the residual soil behind the GBS and not by the movement of the GBS wall. The higher horizontal pressure from the backfill induces the movement of the GBS wall and resulted in larger deformation. Figure 12 shows the maximum pull-out force developed in geogrids which is constant with time.

Figure 12 Tensile /Pull-out force in geogrids

4. FIELD VERIFICATION

The GBS walls at the pilot site were exposed to extreme rainfall that started at 2:20 pm on 18th January 2017 after more than 2 weeks of dry period. The intermittent rainfalls lasted until 24th January 2017. The plot of hourly rainfall during this period is shown in Figure 13. The cumulative rainfall during this 1-week period was 204 mm while the highest hourly rainfall was 38.7mm/h at 10:00am on 23rd January. The rainfall caused a rise in GWL causing that part of the gravel sump below the GBS to be submerged with water for about seven hours before it was redistributed to the surrounding soil.

Figure 13 Hourly rainfall on 18th – 24th January 2017

Figure 14 shows the PWP and volumetric water content (VWC) variations recorded by tensiometers and soil moisture sensors. As shown in Figure 2, all tensiometers were located at 2 m depth from crest. Each tensiometer measured the response of different materials (TM1, TM2, TM3, and TM4 measured the PWP in the residual soil, compacted soil, coarse RAP and fine RAP, respectively). The PWP recorded by TM1 and TM2 in compacted soil was between -18 and -22 kPa with very small variation regardless of rainfall infiltration. This shows that the rain-water did not seep through the GBS wall even under extreme condition. The PWP variations in coarse grained layer were almost constant at -4.5 kPa. This indicates that there was no breakthrough under the extreme rainfall from 18th to 24th January. The PWPs recorded in fine grained material were slightly affected by rainfall. The PWPs only fluctuates between -14 and -17.7 kPa. Even though the PWP in fine RAP fluctuated due to rainfall, the maximum recorded VWCs during this extreme rainfall period (18th – 24th January 2017) were still less than the saturated volumetric water content (θ_s) of the material, thus water flow through these layers was very small if not impossible.

Figure 14 PWP and VWC variation in GBS wall subjected to extreme rainfall on 18th – 24th January 2017

The PWP recorded in each material and the corresponding VWCs at 2 m depth are plotted in the SWCC graphs (Figure 4 and Figure 6). For the compacted soil, the points are closer to the wetting curve (Figure 4). For coarse RAP, the points are plotted close to the residual condition of the SWCC curve of coarse RAP (Figure 6), while for fine RAP, the points are plotted along the drying curve. The less than saturated condition in fine grained layer is due to the presence of ASM bags over the bags of fine materials, thus rain water was retained by the ASM before it reached the fine RAP. There was no field measurements of PWP and

VWC in the ASM bags. As shown in Figure 6, the soil-water retention of the ASM is highly hysteretic, thus could store large amount of water. The coefficient of saturated permeability of the ASM is low (1×10^{-5} m/s), thus water flow slowly through the ASM layer.

Figure 15a shows the total vertical and horizontal pressures recorded by earth pressure cells in GBS wall at 1.5 m depth from the crest. The pressures were normalized to the reading on 16th January 2017 (the beginning of this rainfall period). The total vertical and horizontal pressures in the GBS wall increased slowly due to the increase in the unit weight of the wetting soil. Similar with the numerical results (Figure 11), the increase in the total horizontal pressure was more than the increase in the total vertical pressure. However, the increase was not as significant as evaluated in the numerical analysis because the field study has been subjected to many periods of rainfall since the construction and the start of monitoring period on 1st July 2016. The normalized total vertical and horizontal pressure with respect to the initial reading on 1st July 2016 are shown in Figure 15b. The figure shows that total horizontal pressure increased by almost three times the initial reading while the total vertical reading only increased slightly due to rainfall but decreased again after the rainfall stopped. This explained the change in deformation pattern of the GBS wall as shown in Figure 9a whereby the top part moved quite significantly due to the weight of rain-water collected in the ASM and fine RAP.

Figure 15 Transient total vertical and horizontal earth pressure recorded by Earth pressure cells normalized with (a) reading on 16th Jan 2017 (b) initial reading on 1st July 2016.

5. CONCLUSIONS

The conclusions from this study are as follows:

1. The pore-water pressure variations in the reinforced (compacted) soil from numerical analysis are in agreement with those from tensiometer readings. The pore water pressure in the compacted soil was constant even during the extreme rainfall periods, showing that the compacted soil was protected from rainfall infiltration and seepage.
2. Stability analysis showed that the local factors of safety were constant during the rainfall periods, indicating that the extreme rainfall did not affect the local stability of the GBS wall.

3. The changes in the deformation pattern from the numerical analyses agreed with the field observation of earth pressure; which indicated that rainfall infiltration affects the horizontal pressure more than the vertical pressure in the top part of the GBS wall.

6. ACKNOWLEDGEMENT

The findings published in this paper are based on the project Geobarrier System for Use in Underground Structure (Grant No. SUL2013-3) that is supported by a research grant from the Ministry of National Development Research Fund on Sustainable Urban Living, Singapore.

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