A case study of reinforced soil slopes in Ontario

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ABSTRACT
This paper presents a case study of reinforced soil slopes (RSS) along Bathurst Street in the Region of York, Ontario. Six RSSs were designed to be up to 10 m in height and reinforced with geogrids at vertical spacings of 0.6 m for the roadway embankments over three culverts. The slope angle was 70º and slope face units consisted of vegetated geocells. Two years after the RSS construction, the face units of one RSS collapsed after a significant rain event. Field investigation and global stability analysis were carried out. It was found that the possible failure mechanisms of the RSS were the infiltration rate of surface water exceeded the drainage capacity of the RSS and the buildup of hydrostatic pressure exceeded the sliding resistance between the geocells. The failed face units were then reinstalled using the same geocells with the installation of a vertical drainage layer behind the geocells. Following the reinstallation of the face units of the RSS, ground movement monitoring was conducted for all RSSs over the culverts. Field inspection was carried out one year after the reinstallation of the face units. It was found that all RSSs were globally stable and reinstalled face units were stable. However, localized soil erosions and geocell displacements were observed at the face units of remained five RSSs. Improvements for the design and construction of the RSS are recommended.

RÉSUMÉ
Cet article présente une étude de cas sur les pentes de sol renforcées (RSS) le long de la rue Bathurst, dans la région de York, en Ontario. Six RSS ont été conçus pour être jusqu'à 10 m de hauteur et renforcés avec des géoréseaux à des espacements verticaux de 0,6 m pour les remblais de la chaussée sur trois ponceaux. L'angle de pente était de 70o et les unités de face de pente se composaient de géocellules végétalisées. Deux ans après la construction du RSS, les unités de visage d'un RSS se sont effondrées après un événement pluvieux important. Des enquêtes sur le terrain et une analyse de la stabilité mondiale ont été effectuées. Il a été constaté que les mécanismes de défaillance possibles du RSS étaient le taux d'infiltration de l'eau de surface dépassait la capacité de drainage du RSS et l'accumulation de pression hydrostatique dépassait la résistance coulissante entre les géocellules. Les unités de face défaillantes ont ensuite été réinstallées à l'aide des mêmes géocellules avec l'installation d'une couche de drainage vertical derrière les géocellules. À la suite de la réinstallation des unités faciales du RSS, la surveillance des mouvements au sol a été effectuée pour tous les RSS sur les ponceaux. L'inspection sur le terrain a été effectuée un an après la réinstallation des unités de face. Il a été constaté que tous les RSS étaient stables à l'échelle mondiale et que les unités de visage réinstallées étaient stables. Toutefois, des érosions localisées du sol et des déplacements géocellulaires ont été observés aux unités de face de cinq RSS. Des améliorations pour la conception et la construction du RSS sont recommandées.

1   INTRODUCTION

Roadway embankments approaching a culvert or bridge are often constrained by retaining walls due to limited space. Flexible retaining walls such as reinforced earth wall and reinforced soil slope (RSS) are generally used as they are tolerant of the ground settlement under the weight of embankment fill. However, if the face units of the reinforced earth wall and RSS are not properly designed and constructed, the serviceability of the roadway could be significantly affected.

This paper presents a case study of RSSs along Bathurst Street in the Region of York, Ontario. Six RSSs were designed to be up to 10 m in height and reinforced with geogrids at vertical spacings of 0.6 m for the roadway embankment over three culverts. The slope angle was 70º and slope face units consisted of vegetated geocells. Two years after the RSS construction, the face units of one RSS collapsed after a significant rain event. Field investigation, global stability analysis, and sliding stability of face units were carried out. It was found that the possible failure mechanisms of the RSS were the infiltration rate of surface water exceeded the drainage capacity of the RSS and the buildup of hydrostatic pressure exceeded the sliding resistance between the geocells. The failed face units were then reinstalled using the same geocells with the installation of a vertical drainage layer behind the geocells. Following the reinstallation of the face units of the RSS, ground movement monitoring was conducted for all RSSs over the culverts. Field inspection was carried out one year

after the reinstallation of the face units. It was found that all RSSs were globally stable and the reinstalled face units were stable. However, localized soil erosions and geocell displacements were observed at the face units of remained five RSSs. Improvements for the design and construction of the RSS are recommended.

2 SITE CONDITIONS AND REINFORCED SOIL SLOPE CONSTRUCTION

The reconstruction of Bathurst Street between the Township of King and Town of East Gwillimbury in the Region of York, Ontario involved extensive changes to both the vertical and the horizontal alignments for about two kilometers of the road. Up to about ten-meter deep cuts and ten-meter high embankment fills were required. A number of culverts were constructed for wildlife passages and creeks/storm water. Three corrugated steel pipe arch (CSPA) culverts (A, B and C) with dimensions of 7.67 m x 4.55 m are discussed in this paper.

A geotechnical investigation (by others) including borehole drilling, standard penetration tests (SPT) and laboratory testing of soil index parameters was carried out to support the design of the embankment and culverts. At Culvert A, the boreholes encountered 5.4 to 5.5 m very loose to compact gravelly sand to sandy silt fill overlying very stiff clayey silt till. The groundwater level was measured from the monitoring well at a depth of 2.9 m below the existing ground surface. At Culvert B, the boreholes encountered 1.5 to 5.5 m of very loose to compact silty sand to sandy silt fill overlying compact to very dense sandy silt to silty sand or sandy silt till. The groundwater level was measured from the monitoring well at a depth of 4.7 m below the existing ground surface. At Culvert C, the boreholes encountered 2.1 to 2.9 m of very loose to loose sandy silt or very soft to stiff clayey silt fill overlying compact to very dense sandy gravel, company to very dense sandy silt till, or stiff to very stiff clayey silt till. The groundwater level was measured from the monitoring well at a depth of 3.3 m below the existing ground surface. At these three locations, organic material including decayed wood was encountered in the fill. The water contents were 7% to 28% for the sandy silt to silty sand fill, 31% to 103% for the organic material, 9% to 21% for the sandy gravel to sandy silt, 11% to 32% for the sandy silt till, and 17% to 21% for the clayey silt till. The geotechnical report recommended a SLS (serviceability limit state) bearing capacity of 150 kPa and factored ULS (ultimate limit states) resistance of 225 kPa for a design of a mechanically stabilized earth wall or reinforced soil slope (RSS).

Based on the soil bearing capacity and parameters recommended in the geotechnical investigation report, a RSS was designed for the wing walls at each CSPA culvert. A typical cross section of the RSS is shown in Figure 1. The RSS consists of:
1) Backfill made of OPSS (Ontario Provincial Standard Specification) Granular B material.
2) Geogrid reinforcements placed at 0.6 m vertical spacing. The reinforcement was placed continuously throughout the embedment length of

8 m. For the top 6 m of embankment, the geogrid ultimate limit strength is 52.5 kN/m, creep limited strength is 33.9 kN/m, and long-term design strength is 28 kN/m. For the lower 4 m of the embankment fill, the geogrid ultimate limit strength is 73 kN/m, creep limited strength is 47.1 kN/m, and long-term design strength is 38.9 kN/m.
3) Geo-web face units supported by 0.6m wide and 0.4m high concrete slabs.
4) The gradient of the slope was 70°. Top 0.9 m of the embankment was designed as unreinforced fill with a 1V:3H outer slope. However, based on the information provided by the contractor, the top layer of the geogrid was not installed, so that the actual height of the unreinforced section of the embankment was 1.5 m as constructed.
5) The RSS was designed to be up to 9.4 m in height at Culvert A, up to 9.0 m in height at Culvert B, and up to 10 m in height at Culvert C.

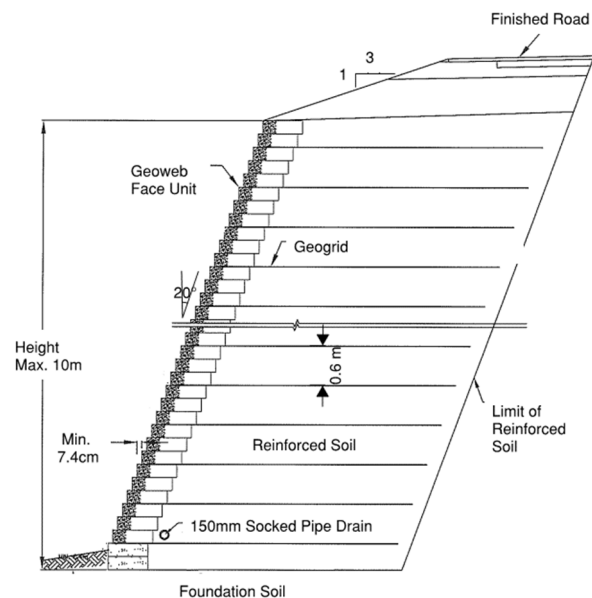The RSSs were constructed in the summer of 2015.

Figure 1 A typical section of designed reinforced soil slope

3 SLOPE FAILURE AND REVIEW OF REINFORCED SOIL SLOPE DESIGN

Following the period of heavy rains in late April and early May of 2017, the RSS at Culvert A failed in early May 2017. As shown in Photo 1, the failure feature was concentrated around the culvert, and that the base of the failure surface was at a depth of approximately 6 m from the top of the embankment. The photo also shows that the scarp of the slope was at the inner edge of the shoulder area. The pavement structure of roadway was largely unaffected by the failure. However, a number of cracks were observed on the road shoulders at Culvert A as well as at Culverts B and C built using the same method and material as Culvert A. Movement of about 100 mm was observed on the top of RSS at Culvert 3.

Photo 1. Failure of reinforced soil slope at Culvert A

Following the failure, the stability of the RSS and the sliding stability of geo-web facing units were reviewed. Field investigation and laboratory testing were also carried out.

3.1   Slope stability analysis

Slope stability analyses were conducted for circular slip surfaces using commercial limit-equilibrium software. The analyses were done for as-built state, i.e. without geogrid reinforcement along top 1.5m of the embankment.

Two groups of slope stability analyses were done:
1) The global analysis of the slip surfaces corresponding to the whole slope with a variable water table from 1 m below the RSS (i.e. dry condition) to the top of RSS (i.e. fully saturated condition); and
2) Localized analysis of the slope, with the crest of the slope corresponding to the shoulder area and slope's toe at approximately 6 m depth.

The slope analyses were performed using three different slice methods: Bishop simplified method, Generalized Limit Equilibrium (GLE) method and Spencer method. The long-term design strength was used for the geogrid in the analysis. The geogrid tensile strengths were taken as 38.9 kN/m and 28 kN/m for lower 4 m and top 6 m of the embankment fill, respectively. A distributed load of 12 kPa was placed on the top of RSS to simulate the traffic load. The soil parameters used in the slope stability analysis are listed in Table 1. For the geo-web face unit, the friction angle for fill material was taken as 30° and the strength of geo-web was assumed as 5 kPa in the global stability analysis.

Table 1. Soil Parameters for Slope Stability Analysis

Soil Type	Unit Weight (KN^3/m)	Friction Angle ($^{\circ}$)	Cohesion (kPa)
New Fill	21	32	0
Clayey Silt Till	21	30	10
Geo-web Face Unit	19	30	5
Concrete Base Slab	24	0	1000

Bishop simplified method give the lowest factor of safety (FOS) for the global analysis of the RSS among the different slice methods. Figure 2 shows a FOS of 1.192 when the water table rises to the middle level of RSS using Bishop simplified method. Figure 3 shows the FOS decrease from 1.5 for the water table at 1 m below the RSS to 0.72 for the water table at the top of the RSS. When the 70% of the RSS is below the water table, the RSS will be unstable with a FOS less than 1. This means the global stability of the RSS is significantly affected by the water table.
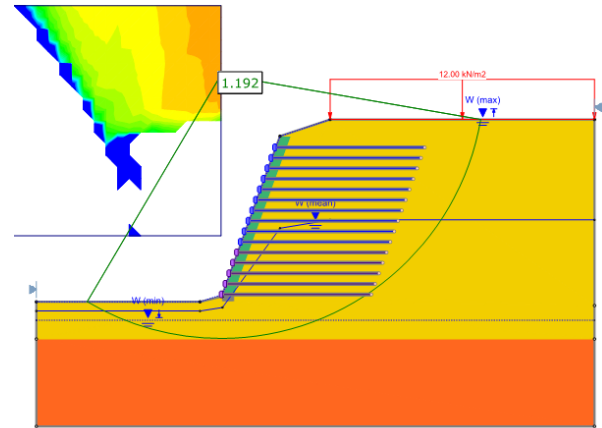

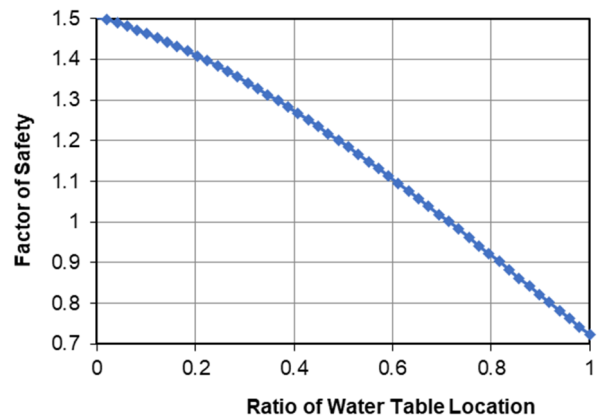Figure 2. Global stability of RSS with middle water level

Figure 3. Variation of factor of safety with ratio of water tale location (0 for lowest water table and 1 for highest water table)

The localized analysis of the slope in dry conditions produced the minimum FOS of 1.789 as shown in Figure 4 using Bishop simplified method. The other two methods gave the FOS of 1.778 to 1.779. The value of FOS is affected by the variation of geotextile strength. If the geogrid tensile strength reduces to 14 kN/m, half of the long-term design strength, the FOS reduces to 1.31. If the geogrids are installed at a vertical spacing of 1.2 m, the FOS reduces to 1.18. These indicate that the RSS failure was unlikely due to the reduction of geogrid strength with time or less installation of geogrid layers.
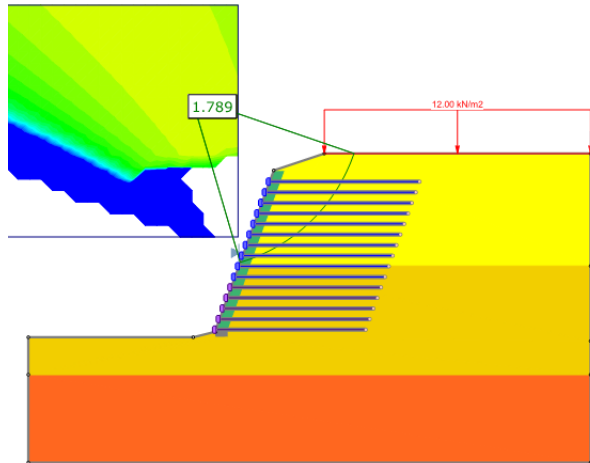

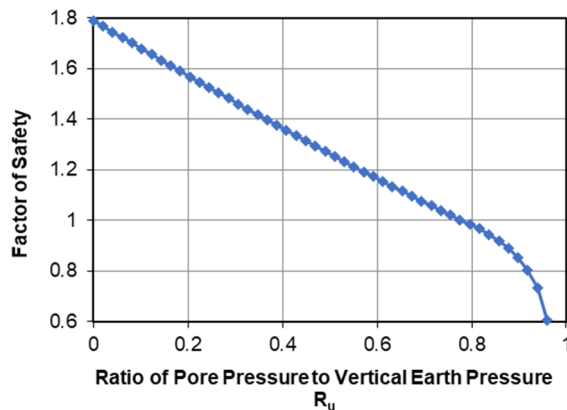Figure 4. Localized stability of RSS in dry conditions

Figure 5. Variation of factor of safety with pore water pressure

The pore water condition at the top section of RSS can be modelled using a parameter, R_u defined as ratio of pore water pressure to the vertical earth pressure. If R_u is equal to 0, the soil is dry with zero pore water pressure. When R_u is greater than 0.5, the soil becomes fully saturated. Figure 5 shows FOS decrease from 1.79 to 1 when pore water pressure increases from 0 to 0.776 of the vertical earth pressure. This means that pore water pressure building

from the top, e.g. by ponding of water at the top of relatively low permeable layer, will lead the local failure of the RSS.

3.2   Sliding stability of geo-web facing units

The infiltration of surface water affects the horizontal stability of the facing units. Geo-web facing unit is HDPE cellular confinement system which were filled with either granular material or topsoil. Each facing unit was approximately 1.6 m long, 0.6 m wide and 0.2 m high. Based on the presented drawings 80% of the volume was filled with granular material.

The facing units have very low permeability in horizontal direction, and limited resistance to sliding in case of hydrostatic pressure buildup. Assumed that the soil and geogrid comprise a stable system which does not transfer any lateral load to the facing units, the face units will be loaded only by the hydrostatic pressure which could build up through ponding of water behind the wall. The factor of safety against sliding of a stack of facing units of height, h can be calculated as follows:

$$FOS = \frac{b(\gamma h - \gamma_w h_w) \tan \phi}{0.5 \gamma_w h_w^2} \qquad [1]$$

where b is the width of face unit, taken as 0.6 m; γ is the unit weight of the soil material within cellular confinement, taken as 19 kN/m³; h is the height of face units; r_w is the unit weight of water; h_w is the height of water behind the facing; and ϕ is the friction angle between the face units, taken as 30°.

For a 6 m high stack of facing units, the factor of safety against sliding for various h_w/h is presented in Figure 6. The 6 m high facing will become unstable if there is a buildup of 2.4 m of hydrostatic pressure behind the wall.
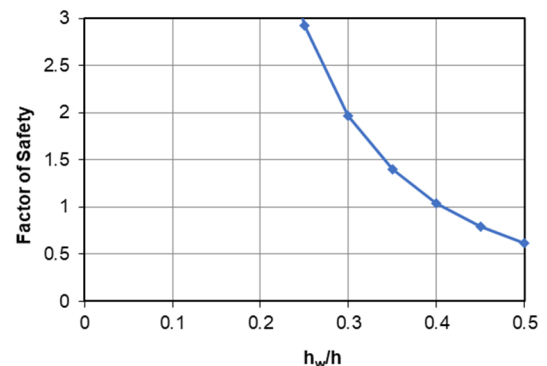

Figure 6. Variation of factor of safety with water head

3.3   Field investigation and laboratory testing

During the reconstruction of the failed section, it was found that some of the geogrid sheets were omitted at Culvert A.

In attempt to verify the presence of geogrid panels within the RSS, sonic drillers to core through the embankment were carried out in Culvert B. The sonic drilling, which turned out to be rotary drilling, didn't quite produce the clear answer as expected. Following a

thorough laboratory review of core samples, only 60% to 70% of designed geogrid panels were found in two boreholes and 47% in one borehole. However, the contractor argued that they could prove that all the geogrid panels had been installed exactly to the design. As discussed previously, the RSS is globally stable even if only 47% of designed geogrid panels is installed.

Grain size analyses were conducted for seven samples of the embankment fill taken during the borehole drilling. The tested samples contained 34% to 56% gravel, 33% to 59% sand, 7% to 12% fine size particles. This confirms that the backfill can be generally considered as free drain material. However, relatively low permeable layers could exist in the granular fill with higher fine content.

A set of three direct shear tests were conducted and found the friction angel for the backfill was 37°, which is higher than that used in the global stability analysis.

3.4   Discussions

The analyses of a number of possible failure mechanisms identified the most likely scenario behind the RSS wall failure as follows:
1) Excessive amounts of surface water runoff from the road spilt over the top of the RSS;
2) The infiltration rate of surface water exceeded the drainage capacity of the RSS;
3) The exceedance caused buildup of hydrostatic pressure on the back of the face units;
4) Top-down buildup of the hydrostatic pressure gradually exceeded the sliding resistance between the face units; and
5) The face units pushed the RSS to fail.

Based on the analysis, remediation measures were recommended as follows:
1) Building of a concrete or asphalt curb and formation of a shallow swale along the line of the guard rails to prevent future spill. The curb should be high enough to contain the maximum estimated runoff.
2) Installation of soil nails to establish physical connection between the face units and the geogrid reinforcement in order to improve sliding resistance.

The first recommendation was accepted. The second recommendation was not accepted and replaced by movement monitoring of the RSSs using inclinometers and survey targets at the three culverts.

4   MOVEMENT OF REINFORCED SOIL SLOPES

Monitoring of RSS movements at Culverts B and C was conducted from July 14, 2017 to August 30, 2018 using eight inclinometers installed on the shoulders (2 on each side of Culverts B and C) and eighty survey targets installed on the face of walls (20 on each face of walls).

The inclinometers were installed 5.5 m below the ground surface. The maximum lateral displacement was 3 mm at Culvert B and 5 mm at Culvert C for 15 months monitoring period. Based on these results, it was considered that no significant lateral movement had

occurred at Culverts B and C, and the global stability of the RSSs as well as the embankment at Culverts B and C is satisfied.

Twenty survey targets (Nos. 201 through 220) were installed on the face of east RSS at Culvert B at upper, middle and lower levels. It is not clear for the installation depth/length of the survey target. If the installation depth/length of the survey targets is less than the frost depth of 1.4 m, the monitoring data will be sensitive to the frost. The lateral movement measured in 15 survey targets was less than 10 mm, whereas lateral movement measured in 1 survey target (No. 214) installed at middle level and 4 survey targets (Nos. 209, 211, 213 and 216) installed at upper level ranged from 12 mm to 63 mm. The vertical settlements measured in these 5 survey targets ranged from 17 to 119 mm. It was noted that the lateral movement and vertical settlement did not increase linearly. For example, the lateral movement and settlement measured at No. 13 were 1 mm on June 21, 2017 and increased to 51 mm for lateral movement and 119 mm for settlement on June 27, 2017. From June 21 to August 29, 2018, the additional lateral movement and settlement were only 11 mm and 18 mm, respectively. This suggests that the lateral movement and settlement measured by the survey targets were likely due to the local erosion of soil at the face units/cells of the RSSs by migration of water through the face units/cells as shown in Photo 2. The lateral movement and settlement measured in the upper survey targets also indicate that consolidation of the fill material inside the face units of the RSS walls.

Photo 2. Local soil erosion/cell movement of west RSS at Culvert B

Twenty survey targets (Nos. 101 through 120) were installed in the face of west Culvert B at upper, middle and lower levels. The lateral movement measured in 16 survey targets was not more than 10 mm, whereas lateral movement measured in 4 survey targets (Nos. 106, 109, 111 and 113) installed at upper level ranged from 13 mm to 32 mm. The vertical settlements measured in these 4 survey targets ranged from 24 mm to 34 mm. It was also

noted that the lateral movement and vertical settlement did not increase linearly.

Twenty survey targets (Nos. 401 through 420) were installed in the face of east Culvert C at upper, middle and lower levels. The lateral movement measured in 19 survey targets was not more than 10 mm, whereas lateral movement measured in 1 survey target (No. 409) installed at upper level was 13 mm. The vertical settlement measured in this survey target was 15 mm. It was also noted that the lateral movement and vertical settlement did not increase linearly.

Twenty survey targets (Nos. 301 through 320) were installed in the face of west Culvert C at upper, middle and lower levels. The lateral movement measured in 16 survey targets was not more than 10mm, whereas lateral movement measured in 4 survey targets (Nos. 306, 309, 311, and 313) installed at upper level ranged from 12 mm to 21 mm. The vertical settlements measured in these survey targets ranged from 16 mm to 39 mm. It was also noted that the lateral movement and vertical settlement did not increase linearly.

Based on the review of monitoring data measured in survey targets, its was considered that the majority of face units/cells of the RSS walls at Culverts B and C were stable with insignificant lateral movements. Significantly lateral movements were only observed locally, probably due to the local erosion of soil of the face units/cells as wall as the consolidation of the fill material inside the face units/cells.

The monitoring for the survey targets installed on the face of east RSS at Culvert A indicated the maximum lateral movement was not more than 12 mm and settlement was not more than 15 mm from October 11, 2017 to August 29, 2018.

Based on the site visit and review of available geotechnical report, design drawings and monitoring data, the following conclusions can be made for the RSSs at Culverts A, B and C as follows:

1) The global stability of the RSSs is satisfied and the RSSs will not be subject to global failure.
2) No settlement of the pavement and embankment was visually observed in January of 2019.
3) The majority of the face units/cells of the RSS walls were stable. No local failure of the face units/cells of the RSS walls was observed in January of 2019.
4) The significant lateral movements measured at few survey targets installed on the face of RSS walls were not unusual for this type of the face units and likely resulted from a combination of factors including
   a) Erosion of soil from the face units/cells; and
   b) Consolidation of the fill material inside the face units/cells of the RSS walls.

5. CONCLUSIONS

From the review of a failed reinforced soil slope, the following conclusions can be provided:
- The global stability of reinforced soil slopes is generally not an issue.

- The failure of the reinforced soil slope at this site was mainly due to the infiltration rate of surface water exceeding the drainage capacity of the RSS and weak connection between face unit and the geogrid. Installation of impermeable layer at the top of reinforced soil slope will prevent excess infiltration. The face unit should be physically connected with the geogrid reinforcement for the RSS design.
- Maintenance of face units for the reinforced soil slope is required if they are not properly vegetated.