Case study on design of a Geosynthetic Reinforced Soil (GRS) retaining wall with a shear key on a layered soil foundation

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ABSTRACT
Geosynthetic Reinforced Soil (GRS) walls have become more popular due to the advantages presented over other types of retaining walls, including flexibility, ease of construction, speed, and cost-effectiveness. As a result of the flexibility of the facing, these walls are more tolerant of settlements on a weak foundation than conventional MSE walls. However, in cases where the allowable settlement is limited to a few inches, such as in a rail track, the wall foundation should be improved using different methods to minimize the settlements from the surcharge.
This paper provides a case study of using a shear key to improve the stability of a GRS wall. The wall is 8.5 m high and 90 m long, reinforced with uniaxial geogrids, located on a mining site in northern British Columbia. A weak interbedded silty sand to sandy silt layer comprises the foundation of the wall. The global stability analysis indicates that this layer reduces the stability of the wall when it is subjected to surcharge loads. Furthermore, this weak layer creates settlements at the face of the wall that is larger than allowed. Study results showed that a shear key consisting of a trapezoidal trench filled with well-graded compacted rockfill material could improve the ground, increase the wall's stability, and reduce the settlement.

RÉSUMÉ
Les murs en sol renforcé géosynthétique (GRS) sont devenus plus populaires car ils offrent des avantages par rapport aux autres types de murs de soutènement, notamment la flexibilité, la facilité de construction, la rapidité et la rentabilité. Du fait de la souplesse du parement, ces murs tolèrent mieux les tassements sur fondation fragile que les murs MSE classiques. Cependant, dans les cas où le tassement est limité à quelques pouces, comme une voie ferrée, la fondation du mur doit être améliorée en utilisant différentes méthodes pour minimiser les tassements dus à la surcharge.
Cet article fournit une étude de cas sur l'utilisation d'une clé de cisaillement pour améliorer la stabilité d'un mur GRS. Le mur mesure 8,5 m de haut et 90 m de long, renforcé par des géogrilles uniaxiales, situé sur un site minier du nord de la Colombie-Britannique. Une faible couche interstratifiée de sable limoneux à limon sableux constitue la fondation du mur. L'analyse de stabilité globale indique que cette couche réduit la stabilité du mur lorsqu'il est soumis à des surcharges. De plus, cette couche faible crée des tassements à la face du mur qui sont plus grands que permis. Les résultats de l'étude ont montré qu'une clé de cisaillement constituée d'une tranchée trapézoïdale remplie d'un matériau d'enrochement compacté bien calibré peut améliorer considérablement le sol, augmenter la stabilité du mur et réduire le tassement.

1 INTRODUCTION

The Geosynthetic Reinforced Soil (GRS) walls have been popularized since they offer advantages over other types of retaining walls, including flexibility, ease of construction, speed, and cost-effectiveness. Previous studies mainly focused on the wall's performance on rigid ground (Michalowski 1998, Helwany et al. 1999, Xiao et al. 2016). It was reported that the stiffness and strength of the foundation can have a significant influence on the wall's overall behaviour (Rowe and Skinner 2001). A weak and highly compressible foundation layer can largely reduce the wall's stability, increase the deformations at the wall facing and base, and increase the strains in the reinforcements (Ezzein and Bathurst 2008, Mirmoradi et al. 2021).

When weak soil foundation deposits are encountered, additional measures should be taken to improve the external stability and decrease the potential for a large settlement. Examples of the measures include staged construction, pre-loading, surcharge loading, and vertical drains to allow consolidation before and during wall construction (Ochiai et al. 2001). A geogrid layer with high strength and long length can also be used at the wall base to increase the external stability (Skinner and Rowe 2005). For deep soft soil foundation of a GRS retaining wall, piles can reduce the creep behaviour of soft soil (Zou et al. 2016).

This paper presents a case study that improves the foundation condition of a GRS retaining wall by adding a shear key at the wall's bottom. The shear key consists of a trapezoidal trench filled with well-graded compacted rockfill material. The retaining wall is 8.5 m high and 90 m long, reinforced with uniaxial geogrids. A weak interbedded silty sand to sandy silt layer comprises the foundation of the wall. This weak layer can potentially reduce the stability of the wall and create a settlement at the face of the wall that is larger than allowed. The analysis and design incorporated theoretical method, limit equilibrium method, and settlement assessment to ensure that the wall's performance meets the design requirements.

2 SITE GEOTECHNICAL CONDITION

The study area is located at a mine in Northern BC. The mine was planning an expansion and upgrading of existing infrastructure and facilities. The planned retaining wall is on the west side of an existing water treatment plant, as shown in the plan view in Figure 1. The retaining wall aims

to raise the ground elevation to the embankment level of the existing water treatment plant.

Two historical boreholes with Standard Penetration Tests (SPT) were completed at the site area in 2011 and 2016. One additional test pit was performed in 2021 to verify the subsurface condition. The locations of boreholes and test pit are also shown in Figure 1.

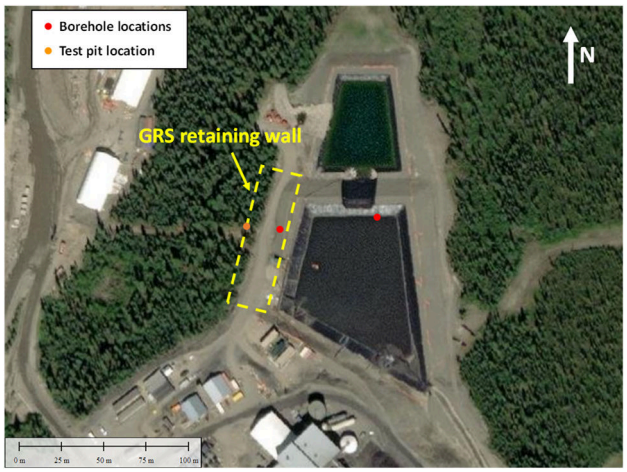

Figure 1. Site plan view showing locations of boreholes and test pit

The subsurface condition revealed by the site investigation is described below. From top to bottom, the soil materials are:

- Topsoil and fill (1.5 m thickness);
- Compact gravel (approximately 1.5 m thickness);
- Compact silty sand to sandy silt (approximately 1.5 m thickness);
- Dense sand and gravel (approximately 3.5 m thickness);
- Very dense silty sand (approximately 6 m thickness);
- Hard clay and silt.

The soil layers exposed in the test pit are shown in Figure 2. The water table was observed at approximately 1 m below the ground surface in the test pit.

3 DESIGN METHODOLOGIES

Various design methods were used to ensure the stability and serviceability of the retaining wall.

3.1 Retaining wall design based on AASHTO 2002

The American Association of State Highway and Transportation Officials (AASHTO) Standard (AASHTO 2002) was used for designing the retaining wall. The calculation was performed using the TensarSoil program. The program assessed the external stability and internal stability of the retaining wall. The external stability included overturning, sliding, and bearing resistance. The internal stability checked the rupture, pull-out, and connection of geogrids. Static and seismic conditions plus flood water level were analyzed in the calculations.

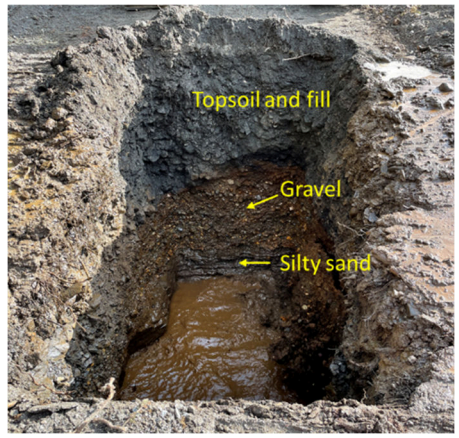

Figure 2. Soil layers observed by test pit

3.2 Limit equilibrium slope stability

The stability of the retaining wall was further evaluated by the limit equilibrium (LE) computer program GeoStudio Slope/W. The LE analysis aimed to investigate the potential deep-seated failure mechanism of the retaining wall. The analysis also included the wall's internal stability.

Morgenstern-Price method (Morgenstern and Price 1965) was selected to compute the Factor of Safety (FoS). A systematic search was performed to obtain the critical FoS from a series of potential slip surfaces. The LE stability analyses were carried out under static and pseudo-static (seismic) conditions. Normal ground water level and flood water level were also considered.

3.3 Settlement evaluation

Settlement analysis on the retaining wall was performed to ensure that the design meets the serviceability requirements in addition to the theoretical and LE analyses. The settlement analysis was conducted using Settle3 software from Rocscience to evaluate the vertical settlement of the foundation.

4 GRS RETAINING WALL DESIGN

The GRS retaining wall is generally 8.5 m high and 90 m long. A pad of 0.65 m thickness is on top of the retaining wall and has a 2H:1V (24°) slope. The facing of the wall consists of welded wire forms filled with rockfill. The rockfill consists of 150 mm minus clean crushed rock. The face of the wall is inclined at 81.5° because of setbacks at each wire form. The reinforced soil is structural fill (maximum particle size of 37.5 mm) and the retained soil is select fill. The structural fill is comprised of 75 mm minus clean, well-graded and free draining sand and gravel material.

The reinforcements are 7 m long uniaxial high-density polyethylene geogrids. Two types of geogrids were specified in the retaining wall. Type A geogrids are from the top to about 3.5 m above the wall bottom. Type B geogrids are for the bottom 3.5 m of the wall. The Type B geogrids have a higher strength than Type A geogrids. The geogrid spacing is 0.45 m, except for the bottom six layers, which are 0.36 m spacing because of connection failure between geogrid and facing wire forms.

A shear key consisting of a trapezoidal-shaped trench filled with rockfill was designed at the bottom of the retaining wall. The purpose was to improve the stability of the wall and bearing capacity of the foundation because of the weak interbedded silty sand to sandy silt layer. The shear key is approximately 3 m deep, with the bottom within the dense sand and gravel. The bottom of the shear key is 1.5 m wide, and the side slopes are 1.5H:1V.

5 RETAINING WALL STABILITY ANALYSIS

This section presents the stability analysis results of the retaining wall. Sections 5.1 and 5.2 describe the soil and reinforcement properties, groundwater and external loads in the analysis. Section 5.3 presents the external and internal stability analysis results based on AASHTO 2002 method. The LE analysis and settlement results are discussed in sections 5.4 and 5.5, respectively.

5.1 Material properties

The soil layers included in the model below the retaining wall are compact gravel, compact silty sand to sandy silt, dense sand and gravel, and very dense silty sand. The embankment of the existing water treatment plant consists of general fill. The reinforced soil is structural fill. The retained soil behind the wall and the pad on the top is select fill.

The soil parameters, including moist unit weight (γ_{moist}), saturated unit weight (γ_{sat}), effective friction angle (ϕ'), undrained shear strength (S_u) and elastic modulus (E), are shown in Table 1. The soil parameter values were estimated by interpreting SPT results and by engineering experience with similar soils.

Table 1. Soil parameters

Properties	γ_{moist} (kN/m^3)	γ_{sat} (kN/m^3)	ϕ' or S_u	E (MPa)
Gravel (compact)	21	22	34°	60
Silty sand/Sandy silt (compact)	19	20	30°	20
Sand and gravel (dense)	22	23	37°	75
Silty sand (very dense)	20	21	33°	30
Clay and silt (hard)	19	20	200 kPa	20
Rock fill	20	21	40°	150
General fill	20	21	35°	-
Select fill	21	22	34°	75
Structural fill	21	22	37°	90

The material properties of geogrids (Type A and Type B) include the internal friction angle (φ_i), tensile capacity (F_t), surface area factor (A), pull out resistance factor (R_p), tensile capacity reduction factor (R_t). The values of geogrid material properties are shown in Table 2.

Table 2. Geogrid properties

Properties	Type A	Type B
Internal friction angle φ_i (°)	26	26
Tensile capacity F_t (kN)	144	175
surface area factor A	2	2
R_p	1.5	1.5
R_t	3.52	3.52

5.2 Groundwater and external loads

The groundwater level was assumed at the toe of the retaining wall and existing ground surface for normal groundwater conditions. This assumption was conservative. Based on flood analysis results, the 200-year floodwater level was assumed at approximately 1.5 m above the toe.

The truckloads were assumed as 50 kPa at least 2 m away from the edge of the wall on the top. The load from equipment and structures was assumed at 100 kPa. The external loads were considered as dead loads.

For the pseudo-static analysis (seismic condition), a horizontal seismic coefficient (k_h) of 0.078g was used, which equals 50% of Peak Ground Acceleration (PGA) in a 1:2475-year return period. The vertical seismic coefficient (k_v) was considered as zero.

5.3 External and internal stability

The top elevation of the retaining wall determines that the wall height is 8.5 m. In the design process, the reinforcements were initially 6 m long (70% of wall height) Type A geogrids and set at a spacing of 0.45 m. However, it turned out that the retaining wall had potential global instability, and the geogrids might experience connection failure for the bottom 3.5 m of the wall. Thus, the final design made the following adjustments: (i) extended the length of geogrids to 7 m; (ii) replaced the bottom 3.5 m of reinforcements with the stronger Type B geogrids; and, (iii) reduced the spacing of the bottom six layers of geogrids to 0.36 m.

The results of external stability are shown in Table 3. The sliding, overturning and bearing verifications all met the design requirements.

Table 3. External stability results

External stability	FoS Static	FoS Seismic	Required FoS
Sliding	2.6	1.9	≥1.5 (static); ≥1.1 (seismic)
Overturning	7.2	4.6	≥2.0 (static); ≥1.1 (seismic)
Bearing resistance	6.2	6.0	≥2.5 (static); ≥1.1 (seismic)

The internal stability results are plotted in Figure 3 below. The plot includes the assessments of tension, pull-out, and connection on the geogrids. The calculations show

that the internal stability of the retaining wall also met the design requirements.

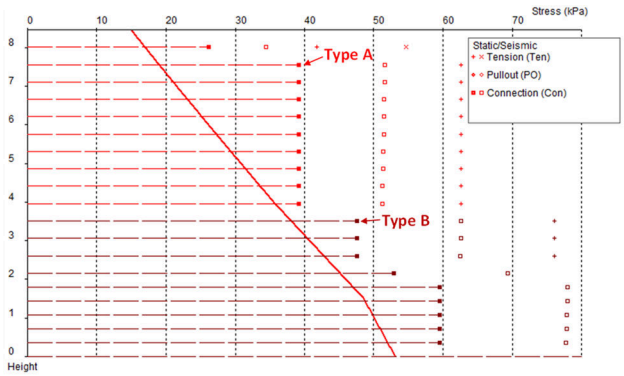

Figure 3. Internal stability results (inclined red line represents horizontal stress at each geogrid layer)

5.4 Limit equilibrium analysis

The limit equilibrium analysis compared the results of global stability of the retaining wall without and with the shear key at the bottom under static, flood, and seismic conditions. The internal stability of the retaining wall was also discussed.

5.4.1 Retaining wall model

The cross-section of the retaining wall model for LE analysis is shown in Figure 4. The geometry of the retaining wall is described in Section 4 above. Several simplifications were made to the model, such as a vertical wall front facing and excluding the facing elements. The Mohr-Coulomb criterion was used for the soil materials.

The geosynthetic material model was used for the uniaxial geogrids in the retaining wall. The bottom 10 layers were Type B geogrids and above that were Type A geogrids.

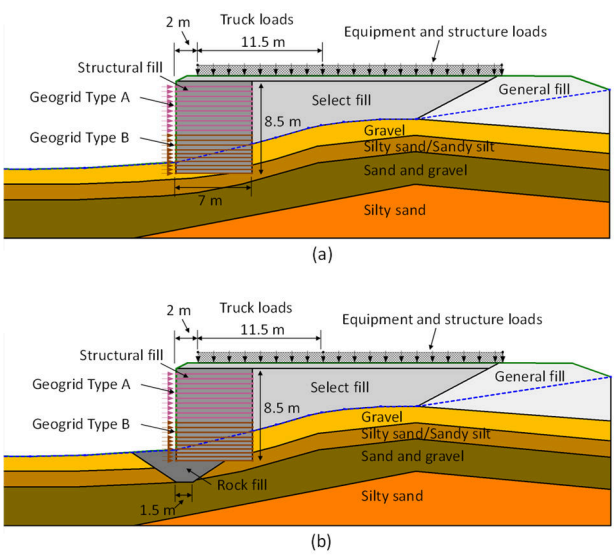

Figure 4. Retaining wall models: (a) without shear key; (b) with shear key

5.4.2 Global stability: with and without shear key

The results of global stability analysis for the retaining wall are summarized in Table 4. The factor of safety (FoS) without the shear key at static condition was 1.4, which is less than the required FoS of 1.5. A circular critical slip surface passed through the weak interbedded silty sand/sandy silt layer, as shown in Figure 5a. The red colored area (FoS<1.5) indicates a wide band of slip surfaces whose FoS were smaller than the minimum FoS requirements.

In comparison, adding the shear key at the bottom of the retaining wall increased the stability to meet the FoS of 1.5 requirements. In this scenario, the critical slip surface was below the shear key bottom and was within the dense sand and gravel layer, as shown in Figure 5b.

The FoS under flood and seismic conditions were greater than the required FoS without or with a shear key (Figure 6). The rising water level at the toe added extra water pressure, thus improving the overall stability. The green-colored area (1.5<FoS<1.6) became narrower than the static case as shown in Figure 6a. The seismic load reduced the wall's stability, as can be seen in the wide red-colored zone (FoS<1.5) in Figure 6b. Nevertheless, the stability of the wall was further improved by adding the shear key at the bottom, as shown in Table 4.

Table 4. Global slope stability results

Scenarios	FoS without shear key	FoS with shear key	Required FoS
Static	1.4	1.5	1.5
Flood	1.4	1.5	1.3
Seismic	1.2	1.3	1.1

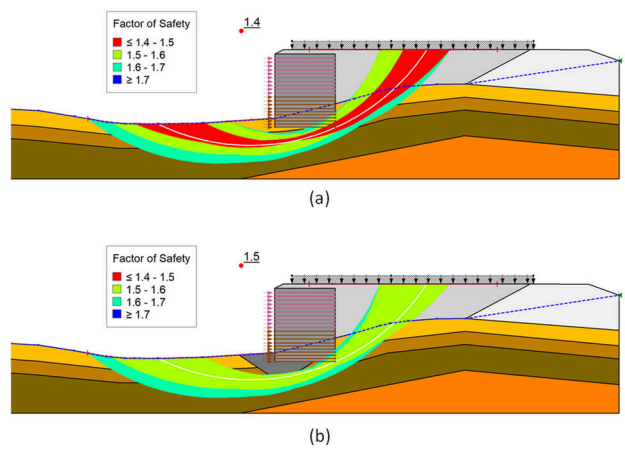

Figure 5. LE slope stability results for static condition: (a) without shear key; (b) with shear key

5.4.3 Internal stability

A summary of the internal stability of the retaining wall using Slope/W is shown in Table 5. The results of internal stability were larger than the required FoS for static, flood, and seismic conditions.

Figure 7a and 7b illustrate the critical slip surface and reinforcement status for static and seismic scenarios. For static condition, majority of slip surfaces had a FoS between 1.8 and 1.9 (red-colored band). The geogrids were governed by tensile resistance. Under seismic condition, the critical FoS reduced to 1.7. The pull-out resistance governed the top layer of geogrid, and the rest geogrids were governed by tensile resistance.

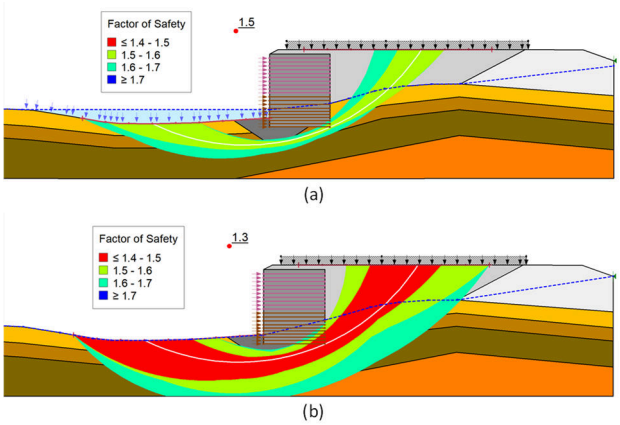

Figure 6. LE slope stability results for (a) flood condition; and (b) seismic condition

It should be noted that the FoS calculated in Slope/W describes the stability of sliding mass under the effect of the reinforcements. The definition of FoS in Slope/W is different from the internal stability in the analytical assessment, such as sliding and rupture of the geogrids.

Table 5. Internal stability results

Scenarios	FoS	Required FoS
Static	1.8	1.5
Flood	1.8	1.3
Seismic	1.7	1.1

5.5 Settlement analysis results

The settlement analysis results of the retaining wall are shown in Figure 8. If the retaining wall was founded on the natural ground without the shear key, the estimated settlement was 59 mm. The weak silty sand to sandy silt contributed to a majority of deformation. As expected, adding the shear key improved the ground and reduced the settlement to 43 mm.

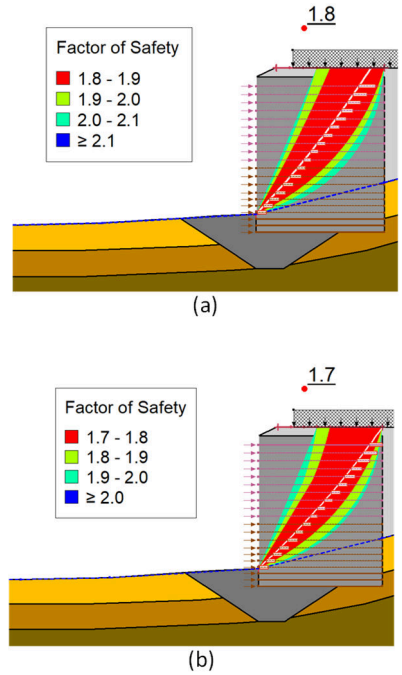

Figure 7. Internal stability of retaining wall under (a) static condition and (b) seismic condition

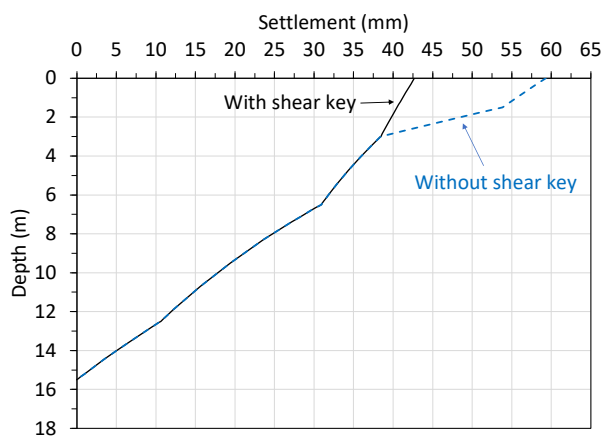

Figure 8. Settlement comparison

6 CONCLUSIONS

This paper presents a case study of using a shear key to improve the foundation condition of a GRS retaining wall. The foundation of the retaining wall consists of a weak interbedded silty sand to sandy silt layer, which affects the retaining wall's stability and settlement. The design procedure includes a theoretical calculation based on AASHTO 2002 design method, a limit equilibrium analysis, and a settlement assessment.

The reinforcement type, length, and spacing were determined by assessing the external and internal stability of the wall using the AASHTO 2002 design approach. The limit equilibrium analysis was performed to evaluate the deep-seated failure mode of the retaining wall as well as

the wall's internal stability. The results indicate that the critical slip surface passed through the weak interbedded silty sand to sandy silt layer when the wall base was on the natural ground. The global stability FoS was less than the design requirement.

A shear key consisting of a trapezoidal trench filled with well-graded compacted rockfill was added at the bottom of the wall to improve the ground at the base of the retaining wall. Results show that the shear key can improve the retaining wall's stability to meet the design requirements and significantly reduce the wall's settlement.

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