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Geogrid reinforced soil walls in Myanmar: an overview

Les Remblais Renforcés en Myanmar: un aperçu

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ABSTRACT: Myanmar is a seismically active country with mountainous terrain and high rainfall. Despite these conditions, geogrid reinforced soil wall system has been widely accepted across Southeast Asia. In remote locations of Myanmar, conventional stone masonry or concrete wall systems are the preferred choices for slope failure mitigation for complex geometries and difficult terrain. As a result, millions of dollars are spent every year on repairing failed slope and failed conventional mitigation. This paper presents an overview of the trials of geogrid reinforced soil wall concept in challenging terrain and subsequent successful implementation of seven geogrid reinforced soil walls with a varying height of 5 to 16 m, along the India-Myanmar Friendship Highway (IMFH).

RÉSUMÉ: Le Myanmar est un pays sismiquement actif avec régions montagneuses et de fortes précipitations. Malgré ces conditions, le système de murs en remblai renforcé par géogrilles a été largement accepté dans tout le Sud-Est de l'Asie. Dans les régions éloignées du Myanmar, la conventionnelle maçonnerie en pierre ou le système de murs en béton sont les choix préférés pour la résolution des glissements de talus dans le cas des géométries complexes et des mauvais sols. En conséquence, des millions de dollars sont dépensés chaque année pour réintervenir sur talus instables et sur ouvrages de soutènement conventionnels qui ont échoué. Cet article présente une vue d'ensemble des essais concernant le concept de mur en remblai renforcé par géogrilles en mauvais sols et de la réalisation de sept murs en remblai renforcé par géogrilles d'une hauteur variable entre 5 et 16 m, le long de l'autoroute India-Myanmar Friendship Highway (IMFH).

KEYWORDS: Keywords: seismic, mountainous, geogrid, Myanmar.

1 INTRODUCTION

Myanmar is the second-largest country in Southeast Asia and occupies the north-western part of the Southeast Asian Peninsula. It is bound to the west by India, Bangladesh, the Bay of Bengal and the Andaman Sea, and to the east by China, Laos and Thailand. Myanmar has three seasons: a dry summer from March to mid-May, a monsoonal rainy season from mid-May to September, and a cool winter season from October to February. Myanmar can be divided into five physiographic regions: the northern mountains, the western ranges, the eastern plateau, the central basin and lowlands, and finally the coastal plains. Central Myanmar is geologically active with a 600 km long fault line that runs north-northwest to south-southeast from Bagan towards Yangon. The topography of Myanmar can roughly be divided into three areas: the Western Hills Region, the Central Valley Region, and the Eastern Hill Region. The steep slopes, the unstable geologic conditions, and the heavy monsoon rains combine together to make the mountainous areas one of the most hazard-prone areas in Myanmar.

Conventional stone masonry or concrete wall systems are the preferred choice for slope failure mitigation for complex geometries and difficult terrain in remote locations of Myanmar. As a result, millions of dollars are spent yearly on repairing failed slope and repairing the failed conventional mitigation.

2 SLOPE FAILURE IN MYANMAR

Myanmar experiences many slope failures depending on the location and the geologic conditions. In mountainous terrain, the main cause of failures are prolonged rainfall and the depth of the weathered profile. In low land areas, the main factors are the slope steepness, scouring, porewater pressure build-up and human activities. Additional factors that contribute to slope failures in both lowland and highland areas include heavy vehicle traffic and other natural triggering factors. In Myanmar, the common practice to repair failed slopes is with concrete walls or stone masonry walls due to the ease of construction, the low cost and because the local authorities are familiar with these methods.

However, because of the limitations associated with these structures, there have been numerous failures. In Figure 1, a typical retaining wall failure is shown. Concrete structures and masonry walls only function as a temporary mitigation measure instead of functioning as a permanent mitigation method. As a result, considerable rework and repair of these structures are needed which in turns, creates an increase in cost and time delays.



Figure 1. Typical retaining wall failure along major roads.

3 TECHNOLOGY OF SLOPE MITIGATION

The most suitable solution that was chosen is the Paramesh hybrid system due to its durability, structural stability, low cost, ease of transport and construction, and its installation speed. Using this system, the wall is permeable which reduces the hydrostatic pressure build-up behind it. The system is structurally able to withstand high deformation due to the seismic actions, to support vehicle loading and to absorb the impact energy from high river velocities. It is an environmentally friendly solution thanks to the fact that the carbon footprint is reduced utilising locally available material and reducing the use of heavy machinery.

The whole system consists of a well-compacted base foundation, a gabion wall facing, steel mesh reinforcement, high strength ParaGrid HF (high friction) reinforcement, backfill material and a drainage composite known as MacDrain. ParaGrid HF geogrids are planar structures consisting of a biaxial array of composite geosynthetic strips characterised by a new geometry to optimise the bonding performance. The strips are composed of a high tenacity polyester a core of longitudinal tendons encased in a polyethylene sheath.

4 KALEWA TRIAL WALL

A trial wall using the Paramesh hybrid system was constructed by the Ministry of Construction near Kalewa in the Sagaing region of north-western Myanmar. The location is situated along the 165 km long India–Myanmar Friendship Road (IMFR) which has been operating for almost 20 years from Tamu–Kalemyo–Kalewa (TKK). This road links India to the second-largest city, Mandalay, and has been under the care of the Myanmar government since 2009. It is an important gateway for the Indian economy since it links to central Myanmar. The area for the mock-up wall is located along river Myittha where the existing concrete masonry wall has collapsed and the toe of the embankment has undergone severe erosion. During the rainy season (from May to October) the river water rises to the crest of the embankment. The two-lane carriageway is reduced to a single lane and remedial work was proposed to extend beyond the original alignment of the road. Considering the urgency of the matter, the client requested that the embankment be repaired within one month. The preliminary investigation had shown that the main reason for the slope failure at this location is due to serious erosion along the bank due to high river velocity and prolonged rainfall. Figure 2 shows the location of the trial wall before the intervention.

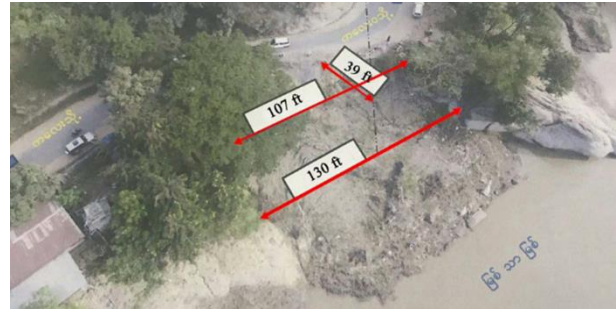


Figure 2. Location of trial wall before intervention.

The construction of the 900 m² surface area wall with 400 m² of scour protection began at the end of April 2018 and was completed by the end of May 2018. The completed trial wall is shown in Figure 3. Upon its successful completion, the trial wall gave enormous confidence to the local authority as well as the Ministry of Construction of Myanmar as to the simplicity of the structure and the economic benefits gained from the exercise. Finally, there is an alternative to a concrete masonry wall which suits the local terrain and provides a long-term solution.



Figure 3. Trial wall after construction.

5 CONSTRUCTION METHODOLOGY

Prior to construction, standard penetration tests (SPT) were carried out by a local soil investigation contractor to determine the properties of the foundation soil up to a depth of 40 ft. The simplified soil model is shown in Figure 4. The construction began with the compaction of the foundation soil with sheepfoot rollers on 24 April 2018 and was supervised by the Ministry of Construction. Dynamic cone penetrometer (DCP) tools were used for evaluating the strength of the soils after compaction.

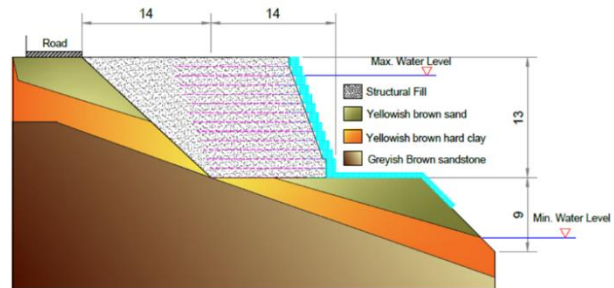


Figure 4. Existing soil layers.

A metallic coated, modular, rock-filled, steel woven wire cage with a tail coupled with high-strength welded geogrid as primary reinforcement was installed at every 1 m spacing until the full height of 13 m wall was reached. The rock filling operation is shown in Figure 5. A roller compactor was used to compact the fill zone at a vertical spacing of 300 mm to obtain 95% of standard Proctor. Selected fill was used as the reinforced soil which complies with the requirement of AASHTO M145 (Standard Specification for Classification of Soils and Soil–Aggregate Mixtures for Highway Construction Purposes). For the Kalewa trial wall, the drainage layer was made by an aggregate layer. At the back of the reinforced soil wall, a 300 mm

size aggregate was wrapped with non-woven polyester geotextile. A layer of rock-filled mattress (known as a Reno mattress) was installed at the toe of the embankment and extends along the designed length as scour protection.



Figure 5. Compaction phase.

6 ABOUT THE SEVEN WALLS.

Following the successful trial wall, the Ministry of Construction commissioned another seven walls in 2019. The walls are located on the highway from Nay Pyi Daw to Chaung Net and are shown in Figure 6.



Figure 6. Location of the seven walls.

Table 1 shows the height of the seven walls constructed along the Nay Pyi Daw to Chaung Net highway during 2019.

Table 1. Wall heights.

Project name	Wall height
Naypyitaw-KanPyar-Magwe Road (Mile 16/1-2)	10 m
Naypyitaw-KanPyar-Magwe Road (Mile 17/1-2)	16 m
Naypyitaw-KanPyar-Magwe Road (Mile 20/3-4)	5 m
Naypyitaw-KanPyar-Magwe Road (Mile 20/5-6)	10 m
Naypyitaw-Chaung Net- Taung Twin Gyi Road (Mile 33/6-7)	10 m
Naypyitaw-Chaung Net- Taung Twin Gyi Road (Mile 36/4-5)	10 m
Naypyitaw-Chaung Net- Taung Twin Gyi Road (Mile 43/6-7)	14 m

Figures 7 to 13 show images related to the seven walls constructed during 2019.



Figure 7. Naypyitaw-Chaung Net - Taung Twin Gyi Road - Mile 33/6-7 (10 m height).



Figure 8. Naypyitaw-Chaung Net - Taung Twin Gyi Road - Mile 36/4-5 (10 m high).



Figure 9. Naypyitaw-Chaung Net - Taung Twin Gyi Road - Mile 43/6-7 (14 m high).



Figure 10. Naypyitaw-KanPyar- Magw Road - Mile 20/5-6 (10 m high).



Figure 11. Naypyitaw-KanPyar-Magwe Road - Mile 16/1-2 (10 m high).



Figure 12. Naypyitaw-KanPyar-Magwe Road - Mile 17/1-2 (16 m high).



Figure 13. Naypyitaw-KanPyar-Magwe Road - Mile 20/3-4 (5 m high).

7 DESIGN METHODOLOGY

This section discusses the aspects related to the design of the structures. The design includes both the sizing of the reinforcement and the drainage system. In particular, the Naypyitaw (17-1/2) wall is analysed, the wall height is 16 m. In Figures 14 and 15, the cross-section of the wall is shown, while the soil properties are shown in Table 2. The structure is reinforced with ParaGrid HF geogrids as primary reinforcement and Terramesh systems as secondary reinforcement. The Terramesh system is characterised by a height of 1 m in the upper layers and 0.5 m in the lower layers to prevent bulging. ParaGrid HF 300 is used at the bottom while ParaGrid HF 150 is used at the superior layers. All the seven walls are located in a seismic area and have a horizontal seismic coefficient equal to 0.2 and a vertical seismic coefficient equal to 0.1. A traffic load equal to 15 kPa is considered for all the structures. The analysis has been done in static, seismic and saturated conditions.

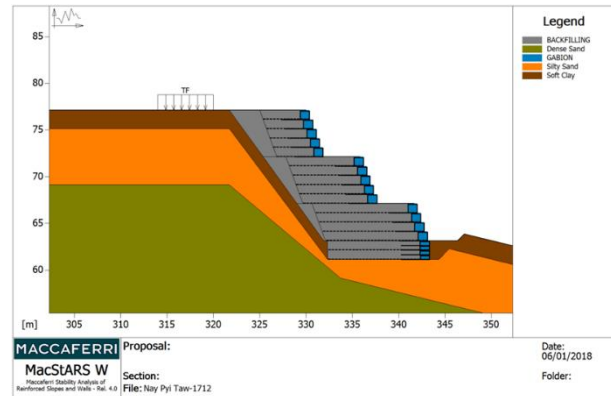


Figure 14. Naypyitaw-KanPyar-Magwe (17-1/2) wall MacStars W cross-section.

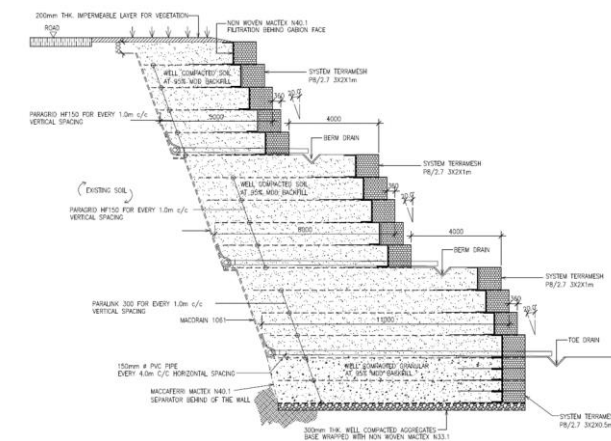


Figure 15. Naypyitaw-KanPyar-Magwe (17-1/2) wall cross-section.

Table 2. Soil properties.

	γ (kN/m ³)	C (kPa)	Φ (°)
Backfill	18	1	32
Dense sand	20	1	37
Gabion	17.5	17	40
Silty sand	18	6	27
Soft clay	16	15	18

7.1 Reinforcement design

The reinforced soil structure is designed using the Maccaferri software MacStars W 4.0 (MACcaferri STability Analysis of Reinforced Soils and Walls). The analysis of the stability conditions is carried out using limit equilibrium methods. The contribution of the reinforcement layers is introduced in the calculation only if they intersect the sliding surface. The tensile strength is mobilised in the reinforcements due to the adhesion between the reinforcement and the material (soil or other reinforcements) that are located above and/or below it. This contribution is simulated with a stabilising force directed towards the interior of the applied point of contact between the sliding surface and the reinforcement. The magnitude of this force is determined by choosing the lower value of the long-term design strength of the reinforcement or the pullout value of the reinforcement.

Whenever the tentative failure surface crosses a reinforcement (see Figure 16), the available force provided by the reinforcement, FR is:

Equations should be numbered consecutively at the end of the line, in parentheses (see Eq. 1). Place a single blank line above and below the equation.

$$FR = \min (F_{breakage}, F_{pol}, F_{por}) \quad (1)$$

where:

$F_{breakage}$ =long-term design strength of the reinforcement.

F_{pol} =pullout force on the left side.

F_{por} =pullout force on the right side.

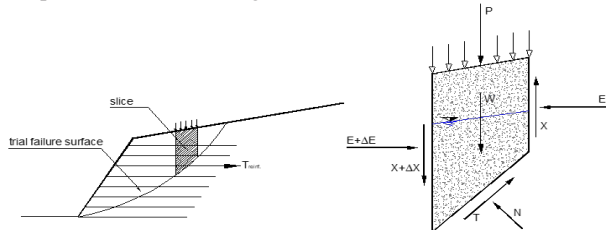


Figure 16. Loads on slices.

7.1.1 Overall stability check

The overall stability check (also called global stability or basic stability) is the stability analysis of a reinforced or un-reinforced slope carried out by using the limit equilibrium method. For design purposes, this stability analysis is required to evaluate the retaining work stability against potential deep-seated sliding mechanisms as well as potential external sliding mechanism to the reinforcing units. Figure 17 shows the global stability in seismic conditions of the structure under study.

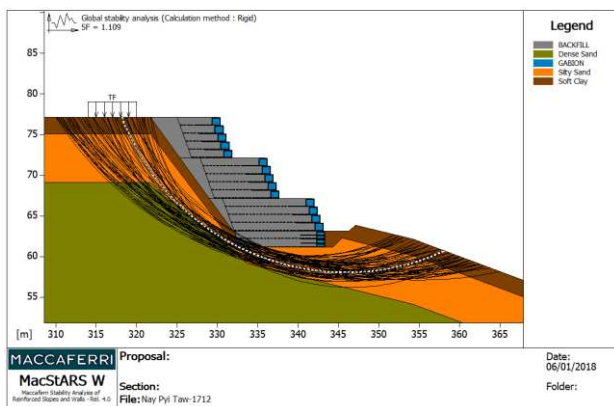


Figure 17. Global stability - Seismic condition.

7.1.2 Internal stability check

The internal stability check (or slope stability) allows the user to determine the design of the retaining structure, more specifically the reinforcing units required (type, spacing between reinforcing unit, length, etc.). According to this type of stability analysis, the surfaces of potential sliding originate from the toe of the reinforcing structure and passes through the reinforcement, and finally terminates uphill. Figure 18 shows the internal stability check in the seismic condition of the structure under study.

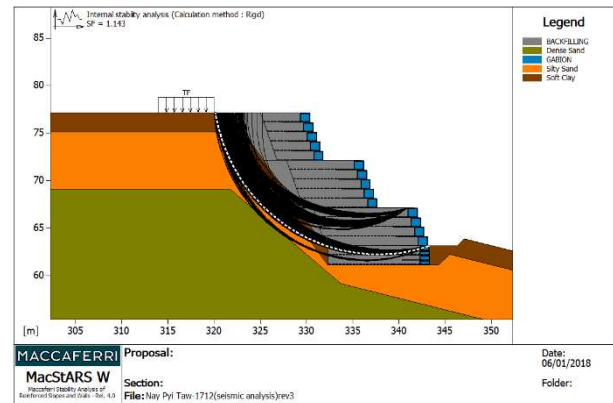


Figure 18. Internal stability - Seismic condition.

7.1.3 Wall stability check

In conducting this type of stability analysis, the entire retaining structure is considered as a monolithic wall consisting of blocks, which form the retaining structure itself.

The stability check of the structure as a retaining wall consists of the three classical stability analysis conducted on retaining walls: overturning, sliding and the foundation bearing capacity. The wall stability checks are shown in Figure 19.

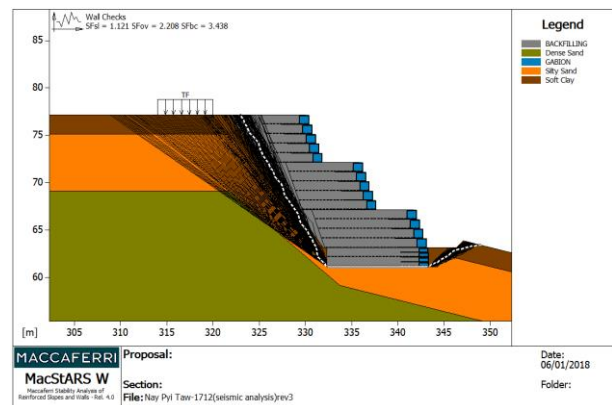


Figure 19. Wall stability check- Seismic condition.

7.1.4 Analysis results

Table 3 shows the results of the analysis. The analysis is run without using any standards (in case of no standard selected, it is common practice to accept a SF higher than 1.5 in static condition and higher than 1.1 in saturated and seismic condition). When performing the seismic checks, a horizontal seismic coefficient equal to 0.2 and a vertical seismic coefficient equal to 0.1 have been considered. The saturated condition refers to the natural soil fully saturated.

Table 3. Safety factors.

		Static condition	Saturated condition	Seismic condition
SF stability	global	1.57	1.28	1.11
SF stability	internal	1.54	1.42	1.14
SF sliding		3.27	3.43	1.12
SF overturning		9.91	12.49	2.21
SF bearing capacity		6.88	3.25	3.44

7.2 Reinforcement design

In this project, the design specifications required a granular drainage layer, e.g. a thick layer of sand/gravel soil. The use of a draining geocomposite instead of the traditional drainage layer is possible by demonstrating that the geocomposite is 'equivalent' to the sand/gravel layer in terms of its hydraulic performance. The MacDrain® geocomposite was placed along the back face of the reinforced soil block.

The problem of the hydraulic transmissivity equivalency between geosynthetics and granular liquid collection layers on slopes has been analysed by Giroud et al. (2000). Such analyses lead to the conclusion that:

- In the thin geocomposite, the hydraulic gradient is constant, and the unconfined free flow surface is practically parallel to the slope.
- In the thick mineral layer, the free water surface is steeper at the front, meaning that the hydraulic gradient is not constant but it is increasing towards the outlet and therefore the actual water flow in the granular layer is higher than the value calculated with a constant hydraulic gradient.

In other words, the flow in a thick granular layer that is used for drainage on a slope is, in most cases, unconfined. In this condition, Darcy's law—which applies to confined filtration flow—cannot be directly used since it provides a lower flow rate than what actually occurs with the unconfined flow. The correct flow rate for unconfined flow can be obtained only by applying a correction factor to the flow rate calculated with Darcy's law. The conclusion is that considering a constant value of the hydraulic gradient, the flow rate in the geocomposite is equivalent to the flow rate in the thick granular layer only if the equivalency coefficient defined by Giroud et al. (2000) is applied. This is the principle of the calculation performed with the Maccaferri MacFlow Studio drainage design software.

The hydraulic and economic comparison is made between the mineral layer with a thickness of 0.3 m and the MacDrain® geocomposites.

The calculation of the available flow rate for the MacDrain® W range affords the results which are illustrated in Figure 20.

Design Calculation		Design Calculation	
Reference water temperature [°C]	20.00	Factor j	0.918
Reference dynamic viscosity [$\rho = 10E-3 \text{ Pa}\cdot\text{s}$]	1.01	Mineral layer thickness needed to carry the flow at the toe [m]	0.300
Design dynamic viscosity [$\rho = 10E-3 \text{ Pa}\cdot\text{s}$]	1.14	Equivalency factor for Tmax (Giroud et Al, 2000)	1.154
Correction for liquid temperature and viscosity	0.88	Calculation of equivalent input flow in MacDrain for the give mineral layer thickness of [m]	0.300
Maximum pressure on MacDrain [kPa]	18.00	Flow rate in the mineral layer (Darcy) [l/s/m]	0.103
Hydraulic gradient	0.342	Flow rate of liquid supply in the mineral layer [l/s/m ²]	0.008
Rain height for design duration [mm]	74.95	Factor j for qh* (Giroud et Al, 2000)	0.918
Rain intensity for design duration [mm/h]	107.07	Equivalency factor for tprescribed (Giroud et Al, 2000)	1.154
Input flow per m2 [l/s/m]	0.008		
Input flow at the toe [l/s/m]	0.118		

Design Input Flow in MacDrain

Design input flow rate for MacDrain [l/s/m]	0.136
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Figure 20. Design calculation.

It must be pointed out that MacDrain® W1041 is equivalent to a granular layer with thickness 300 mm as shown in Figure 21. MacDrain W1061 has been used as a conservative choice.

Specific MacDrain selected is:

MacDrain	Contact	QL	Qa	Safety Factor	Result
W1041	S/S	0.214	0.137	1.008	EQUIVALENT
W1051	S/S	0.273	0.175	1.284	EQUIVALENT
W1061	S/S	0.567	0.363	2.664	EQUIVALENT
W1071	S/S	0.783	0.502	3.679	EQUIVALENT
W1081	S/S	0.953	0.611	4.481	EQUIVALENT
W1091	S/S	1.080	0.692	5.073	EQUIVALENT
W1101	S/S	1.206	0.773	5.667	EQUIVALENT

Figure 21. MacFlow Studio analysis results.

8 CONCLUSIONS

This paper describes the background, design methodology and trials of geogrid reinforced Paramesh hybrid soil walls in Myanmar. Since the Paramesh hybrid system has only been in place for a limited time, there is limited performance data to present. However, due to the cost benefits and confidence in the

Paramesh hybrid soil walls, more walls have been planned for the near future throughout Myanmar.

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