Publication No. 333

6th Asian Regional Conference on Geosynthetics

Geosynthetics for Infrastructure Development

8-11 November 2016, New Delhi, India

PROCEEDINGS

Organised by		Under the auspices of
INDIA Indian Chapter of International Geosynthetics Society	New Delhi Central Board of Irrigation and Power	International Geosynthetics Society

ISBN : 978-8-17-336390-0

2016

Printed by V.K. Kanjlia, Secretary, Central Board of Irrigation & Power
Malcha Marg, Chanakyapuri, New Delhi 110021
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6th Asian Regional Conference on Geosynthetics - Geosynthetics for Infrastructure Development, 8-11 November 2016, New Delhi, India

HIGH HYBRID REINFORCED SOIL SLOPE AS RUNWAY SUPPORT - TANA TORAJA AIRPORT CASE STUDY

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ABSTRACT

The use of hybrid Reinforced Soil Slopes combining anchored gabion units and high strength geogrids for infrastructures development is growing in Indonesia. Hybrid RSS are considered a valid and competitive alternative to other traditional earth retaining structures. Despite this growth, civil engineers and construction companies are not yet familiar with the related design and construction aspects. This paper intends to share the Author's experience regarding the design and the construction of the first stage of a 25 m high hybrid RSS for the new Tana Toraja touristic airport (Sulawesi, Indonesia). It is worth to note that the RSS was built on soft and weathered clay shales and cohesive backfilling materials were used as part of the reinforced soil mass.

This paper aims to present the main technical issues related to the project, the design process, which was carried out using both FEM and Limit Equilibrium method software, and the relevant construction aspects.

1. PROJECT INTRODUCTION

Tana Toraja regency is located in the South Sulawesi province of Indonesia. It hosts every year thousands of tourists coming to admire not only the breath-taking landscapes and the biodiversity of the region, but also the places of the rituals and the ceremonies of the Toraja ethnic group.

Until now, the only way for travelers to approach Tana Toraja was a tough and uncomfortable 8-hours journey by bus or by car starting from the city of Makassar.

For this reason, the Indonesian government, pursuing the tourism development of the area, planned to build a touristic airport in Tana Toraja with a multi-year project starting in 2014 and coming to end in 2017.

2. PROJECT ISSUES

The new Tana Toraja airport runway is 2 km long and approximately 210 m wide, suitable to host ATR type aircrafts. Since a flat surface is required for the construction of the runway, and due to the presence of hills and spurs clashing with the planned runway area, massive cut and fill earthworks have to be undertaken in order to get the required runway elevation.

In the valleys to be backfilled, technically suitable and economically feasible retaining structures have to be built. It is worth to note that the maximum embankment height to be retained is almost 40 m.

The first stage of the Tana Toraja runway construction (2015) involved the design and the execution of a 100 m long and 16 m high retaining structure as runway support. The retaining structure had to be erected in between two hills. Afterward, during the second stage (2016), the retaining structure will be topped with an additional 10 m high structure in order to reach the final runway elevation. The total height of the mentioned structure will be 25 m.

The main technical issues related to the design and the execution of this works were: the high seismicity of the area, the construction to be done during the rainy season months, the presence of clay shale foundation soils and the necessity to use partially cohesive soils as filling material.

During the geotechnical investigation campaign carried out at the beginning of 2015, clay shales were found where the new runway retaining structure was supposed to be erected. Moreover, the foundation area had been previously used as paddy field by local villagers, thus clay shales were already completely saturated. Foundation soil was evaluated unsuitable to bear and carry any structure built on top without any significant deformations or, in the worst case scenario, failure.

Fig. 1: Saturated clay shales

As a project constrain, the runway retaining structure construction had to be started in conjunction with the beginning of the rainy season in the month of December and had to be completed in two months' time. In these months, South Sulawesi statistically experiences the heaviest and most intense rainfall events of the year.

The existing topography and land contour would have directed the water from both the hills to flow directly into the retaining structure foundation area. In order to minimize the water volume flowing in the foundation area, a drainage system had to be carefully planned and executed prior the beginning of the works.

Furthermore, it is well understood that Indonesia is located on the Pacific Ring of Fire, an area with a lot of tectonic shifting, and has to handle the constant risk of earthquakes with a very short return period. According to the Indonesian standard for the design of structures in seismic condition (SNI-1726:2012), Tana Toraja regency is located in an area characterized by a Peak Ground Acceleration with a 500 years of return period equal to 0,3 g.

2.1 Behaviour of Clay Shales

Shale is a fine grained sedimentary rock formed by clays compacted together by pressure in time (Irsyam et al. 2007). Clay shales, when dry and undisturbed, are hard and normally show high shear strength. But, if they absorb water after an unloading process, they can eventually turn to medium or even to soft clay with extremely low strength and poor mechanical properties. Basically, when clay shales are unloaded and exposed to atmosphere, they tend to weather and their mechanical and physical properties change dramatically.

Stark and Duncan (1991) compared the behavior, in terms of shear strength, of soaked clay shales and unsoaked ones. Drained Direct Shear Test had been done on both the soaked and unsoaked sample of clay shale taken from California's San Luis Dam supporting clay layer.

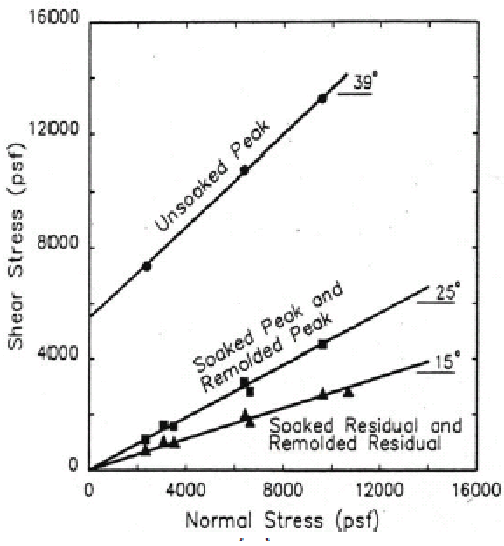


Fig. 2: Shear strength of soaked and unsoaked clay shale (Stark & Duncan, 1991)

Indonesian Ministry of Public Works and the technical committee in charge of the project evaluation and approval, were fully aware of the risk encountered when facing retaining structures built on clay shales. In fact, in 2006, Indonesia experienced in 2006 the massive slope failure of the Cipularang Toll Road embankment which was built over clay shales. This failure attracted national attention due to its strategic and critical function of linking two Indonesian main cities, i.e. Jakarta and Bandung. The Cipularang toll road failure case and the behavior of clay shales has been extensively studied and presented by Irsyam (2007).

For all the mentioned reasons, construction over clay shales generally requires noteworthy care and adequate planning in order to minimize their exposure to weathering agents in order not to compromise structures stability.

3. ADOPTED SOLUTION

A steep unreinforced embankment slope was not a viable solution, instead a very gentle inclination (1:4) and a massive volume of soil were necessary for the unreinforced slope to be stable under the required seismic design condition. Due to space limitation and land acquisition issues, unreinforced embankment slopes were considered an unsuitable solution. Three different types of retaining structures were considered during the planning stages: concrete mass gravity walls, bored piles and hybrid reinforced soil slope (RSS) combining anchored gabion units and high strength geogrids.

The evaluation criteria given by the technical committee for the structure selection were:

- Permeability: the retaining structures shall have a very permeable facing in order to rapidly drain the rainfall waters and to dissipate the hydrostatic pressure developed in the cohesive backfilling soil;
- Flexibility: the retaining structures shall have a flexible behavior in order to accommodate potential differential settlements and to absorb dynamic shocks in case of a seismic event;
- Construction schedule: the structure shall be constructed in 2 months' time;
- Overall cost;
- Local manpower: the retaining structure type and construction method shall maximize the employment of locally available unskilled manpower.

Based on all the above criteria, hybrid Reinforced Soil Slope combining anchored gabion units and geogrids was selected as the best suitable solution under both the technical, economic and social points of view.

The main components of the proposed hybrid RSS were the followings (Figure 3).

Anchored Gabion Units

They are monolithic double twist steel wire mesh box-shaped baskets with a metallic mesh tail extending into the soil in order to reinforce the backfill soil itself. The basket is filled on site with clean-hard stones with suitable dimensions. The double twisted wire mesh is Galfan (Zinc- Aluminium alloy) and PVC coated in order to meet the 60 years of design life requirement imposed by the Ministry of Transportation.

Green Wrap Around Units

They are metallic units made up from a single length of double twisted mesh panel which forms the base, the sloping face and the top part (that is the wrap-around length) of the unit. A bio-degradable blanket is placed behind the metallic sloping face as erosion control against the water superficial flow. A welded steel mesh is pre-assembled to provide rigidity to the face. Bioengineering techniques like live staking, brush layering and hydroseeding can be used to promote a rapid vegetation establishment on the facing.

Both anchored gabion and green wrap around units work as secondary reinforcements and act as fascia providing the local stability at the facing. They ensure that no local mechanism of direct sliding, pull-out or rotational failure can occur.

Geogrids

The primary reinforcing elements consist in high tensile strength strip bonded geogrids manufactured from high tenacity polyester yarn tendons and encased in a polyethylene sheath. The polyester tendons, which confer the required tensile strength, can be manufactured to obtain grades ranging from 100 kN/m up to more than 1300 kN/m. The PE cover guarantees protection against chemical and biological agents and provides UV protection as well as protection against mechanical damages which might occur during installation procedures.

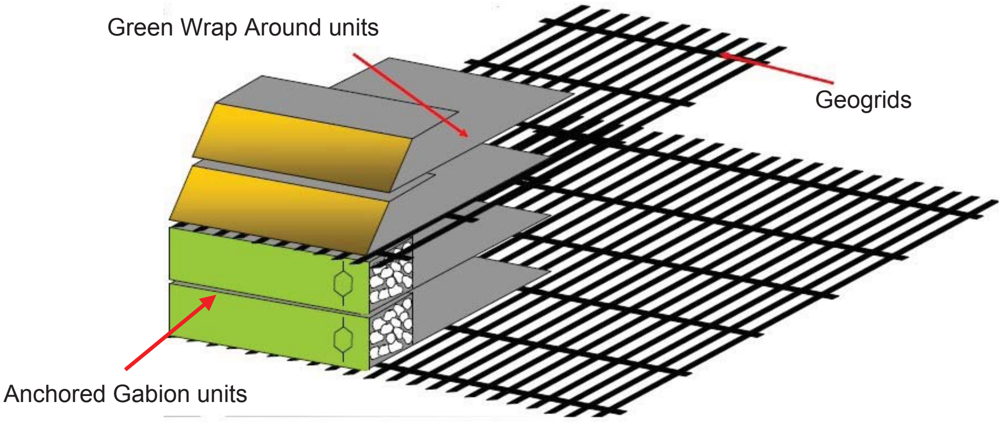


Fig. 3: Hybrid RSS structure components

4. CONSTRUCTION ASPECTS

To anticipate the presence of soft foundation soil, it was considered more convenient to strip out the saturated and soft clay foundation soil and replace it with a selected and compacted soil rather than installing deep bored piles. In fact, bored piles would have not only increased the overall cost, but also dramatically affected the construction schedule.

Before starting the construction, due to the high concern regarding the risk of a slope failure caused by the presence of soft foundation soils, an additional in-situ geotechnical investigation was carried out. 7 (seven) additional Cone Penetration Tests were executed in order to evaluate the soil undrained strength as well as the stripping depth required to reach a stiffer clay layer able to bear the 25 m high structure load.

Undrained shear strength equal to 100 kPa was selected as the minimum acceptable value to be reached at the RSS foundation level. The foundation depth and the soil stripping depth were selected following this criterion. From the CPT results, it was evaluated that the depth of the suitable firm clay shale layer was placed 1.5 m - 2.5 m below the original ground surface, depending on the location.

The location of the 7 (seven) CPTs performed in October 2015 is illustrated in Figure 4.

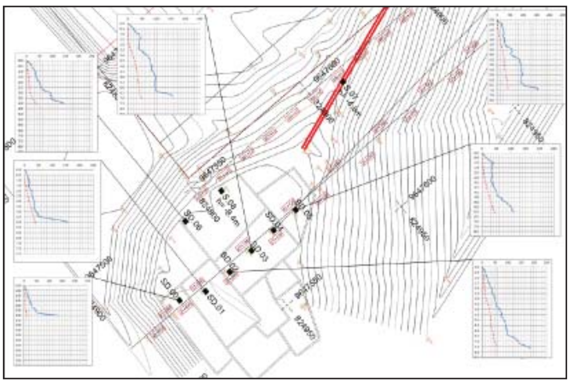


Fig. 4: Plan view and location of the seven CPTs

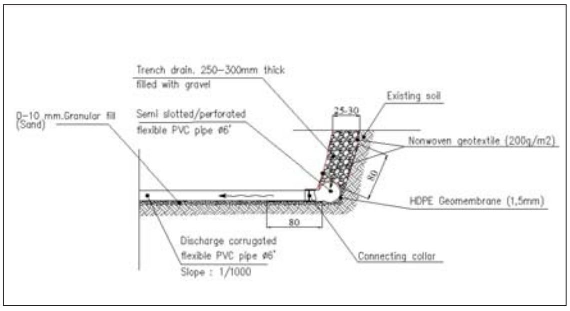
As anticipated, the water flowing from the surrounding hills had to be discharged away from the construction site in order to minimize the contact with the clay layers. To fulfil this target, temporary collecting trenches were excavated at the toe of both the hills.

In addition to this, a permanent drainage system (Figure 5a) was installed at the interface between the existing slope and the new construction RSS. The drainage system consisted in a continuous sloped natural drain realized with round shaped stones (d= 10 – 20 cm) wrapped in two layers of nonwoven geotextile with a filtration function (nominal mass = 200 g/m^2). The average thickness of the drainage layer was 30 cm.

At the bottom of the drainage layer, a perforated PVC pipe was placed for collecting the drained water. The perforated pipe was then connected with orthogonally placed discharge pipes installed every 10 m. The collected water could, in this way, be safely discharged away from the structure.

Underneath the perforated pipes, a 1.5 mm thick HDPE geomembrane was installed for avoiding the water leaching into the foundation soil. This drainage system will keep the soil out of saturated condition. A construction detail of the drainage system is illustrated in Figure 5b.

(a) Permanent drainage system execution



(b) Permanent drainage system detail

Fig. 5

5. DESIGN ASPECTS

As explained, the structure has a temporary height equal to 16 m but the design had to be conducted by considering a final target height equal to 25 m, which will be achieved in 2016 construction stage.

The proposed RSS typical section is illustrated in Figure 6. The 25 m high structure is distributed in 5m-high stepped berms. The first berm, starting from the foundation level, is built using double twist anchored gabion units in order to confer more stiffness to the structure lower part. Meanwhile, for the upper 4 berms, Green Wrap Around units with 60 degrees inclination to horizontal are used. The primary reinforcements are geogrids having an ultimate tensile strength equal to 300 kN/m with an average vertical spacing equal to 1,0 m. The geogrid length ranges from a minimum of 5 m at the top to a maximum of 25 m at the base.

The preliminary design was carried out using a Limit Equilibrium Method based software (i.e. MacStars W 4.0), which has been internally developed by Maccaferri.

Since no information regarding structure deformation and settlements amount could be provided by a LEM analysis, the RSS was checked also using the commercial FEM software PLAXIS 2D. The deviation in terms of overall Safety Factor against failure between the LE and the FEM software was acceptable and less than 5% (Table 3).

The evaluation of the RSS stability under a design seismic condition was mandatory. A pseudo-static model was used to investigate the behavior of the structure under a seismic event causing an additional horizontal mass acceleration equivalent to half of the PGA i.e. equal to $k_h = 0.15$ g.

The minimum required Safety Factors against RSS failure in static and seismic condition were respectively equal to 1.5 and 1.1.

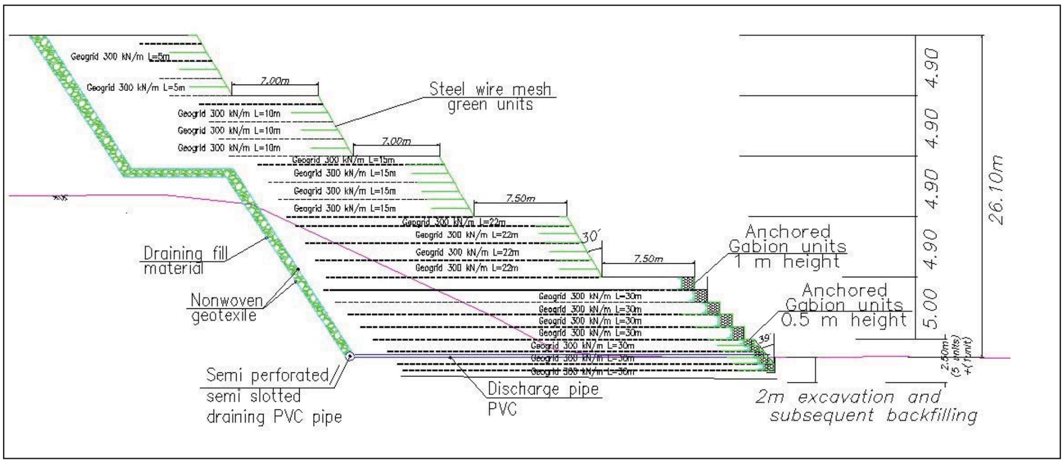


Fig. 6: Typical RSS section

5.1 Material Properties

All the soils were modelled using the well-known Mohr-Coulomb strength model. The relevant soil properties used as FEM analysis input are reported in Table 1.

Table 1: Soil properties used in the FEM analysis

	Name	Type	γ_{unsat} (kN/m^3)	γ_{sat} (kN/m^3)	ν	E_{ref} (kN/m^2)	c_{ref} (kN/m^2)	F (°)	Y (°)
1	Structural Soil	Drained	17.0	17.0	0.30	40000	42.0	16.0	0.0
2	Clay Shale-0	Drained	17.0	17.0	0.33	5000	1.0	10.0	0.0
3	Clay Shale-1	Drained	17.0	17.0	0.33	10000	20.0	17.0	0.0
4	Clay Shale-1b	Drained	17.0	17.0	0.33	10000	20.0	24.0	0.0
5	Clay Shale-2	Drained	18.0	18.0	0.33	25000	20.0	25.0	0.0
6	Clay Shale-3	Drained	19.0	19.0	0.33	35000	20.0	32.0	0.0

It is clear from Table 1 that very low mechanical properties were used for the foundation layer named "Tator Clay Shale-0", which refers to the weathered clay layer exposed to the open environment. The anchored gabion units were simulated by modelling only their rock filling. It is Author's advice not to model the gabion facing using plate elements. In so doing the analysis will result in bending moment been induced in the facing. In reality there will be no moment developed on the facing element (Gouw T.L., 2016). Instead, the correct way is to model the facing element as a soil cluster with the actual dimension of the facing element (Gouw T.L., 2016).

Table 2: Anchored gabion material properties

Name	Type	γ_{unsat} (kN/m³)	γ_{sat} (kN/m³)	ν	E_{ref} (kN/m²)	C_{ref} (kN/m²)	F (°)
Gabion Filling	Drained	17.5	17.5	0.35	40000	12.5	40

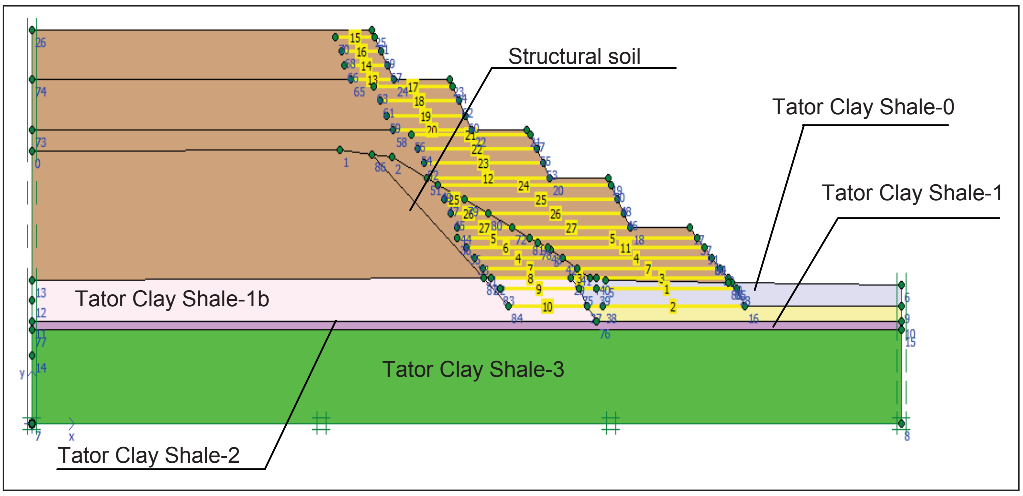


Fig. 7: FEM model

Geogrids are planar reinforcing elements without any bending or compressive stiffness and their main mechanical feature is the tensile resistance. An elasto-plastic model was used to model the geogrids. The geogrid stiffness used in the model was equal to 3900 kN/m. The value is gathered from the geogrid stress-strain curve and considering a geogrid allowable working strain equal to 5%. The ultimate tensile strength of the proposed geogrid is 300 kN/m. The Long Term Design Strength (LTDS) was calculated following the approach proposed by the British Standard BS 8006-1:1995 (Code of Practice for strengthened/reinforced soil and other fills). According to the mentioned Standard, a set of partial safety factors shall be applied to the reinforcement base tensile strength in order to consider the strength reduction in the long-term perspective due to creep, installation damage, and environmental/chemical degradation.

The reduction factors were gathered from a certification of performance issued by a third-party organization (British Board of Agreement) and awarded to the geogrid manufacturer. As a result, the geogrid Long Term Design Strength used in the RSS static analysis was equal to 197 kN/m.

Under seismic condition, since earthquake occur only in a short duration, creep tensile strength reduction need not to be considered in the evaluation of the LTDS (Tatsuoka et Al. 2006), this condition leads to higher LTDS value used in the RSS pseudo-static analysis, which was equal to 272 kN/m.

5.2 Calculation Outputs

The achieved Safety Factors against failure of the RSS typical section illustrated in Figure 6 are reported in Table 3.

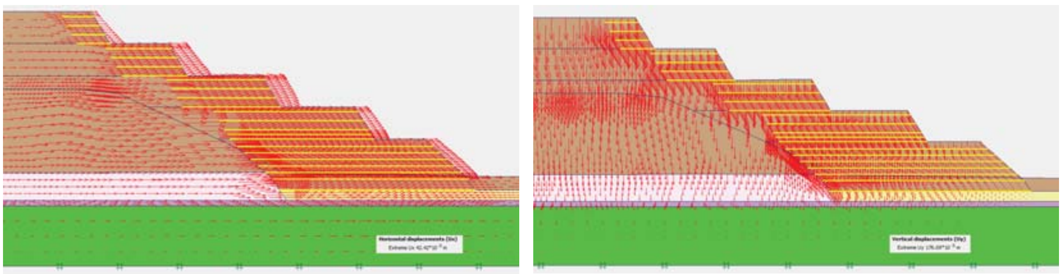
Table 3: Overall stability Safety Factors summary

Condition	Achieved SF with LE software	Achieved SF with FEM software	Minimum Required SF
Static	1.621	1.675	1.5
Seismic	1.097	1.12	1.1

In addition to overall Safety Factors against structure failure, vertical and horizontal post construction and post seismic event displacements were evaluated. In Figures 8a, 8b and 9 the main and relevant software outputs are reported.

The maximum horizontal RSS displacement after the design seismic event was estimated to be less than 5 cm. This value has been considered acceptable and not compromising the structure retaining function.

The maximum vertical RSS displacement after the design seismic event was estimated to be less than 18 cm. This value has been considered acceptable and not compromising the structure retaining function.



(a) Horizontal displacements after seismic event (b) Vertical displacements after seismic event

Fig. 8:

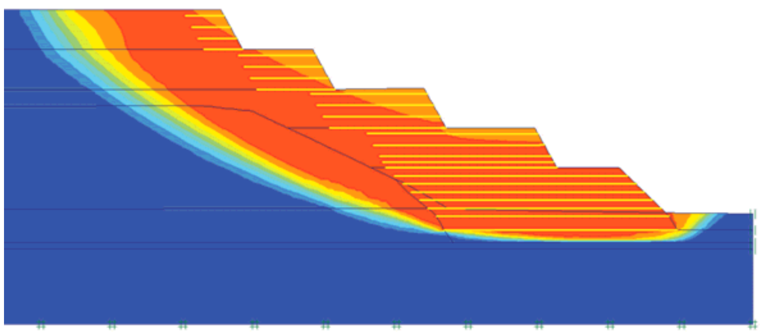


Fig. 9: RSS failure mechanism under seismic event (SF > 1.1)

Based on the design, the actual construction was carried out in December 2015 and finished in January 2016. Despite several issues related to the project, the first stage of the Tana Toraja airport runway construction has been effectively completed within the two months' time frame allowed. Figure 10 shows the completed hybrid RSS structure. Figure 11 shows the structure under construction.

Fig. 10: Tana Toraja RSS, 2 months after end of construction

Fig. 11: Tana Toraja RSS under construction (backfilling and compaction operations)

6. CONCLUSIONS

The paper presented the successful use of hybrid reinforced soil slope (RSS) for a new infrastructure development in Indonesia. Hybrid RSS system combining anchored gabions and high strength geogrids was selected by the Ministry technical committee among other alternative solutions such as concrete mass gravity retaining wall, bored piles and a natural slope. This choice was guided by the system high facing permeability, flexibility, environmental friendliness, cost effectiveness and high involvement of the local manpower for the construction. The use of hybrid RSS allowed the construction of a 100m long and 16m high structure in only 2 months' time despite the severe rainfall events encountered almost every day.

The paper showed that RSS, as long as it is properly designed and carefully executed, can be built also on clay shales and using cohesive materials as backfilling material. An engineered foundation soil and well-planned drainage systems effectively worked as countermeasure to face the weathered and clay shales foundation.

The evaluation of the overall safety factor against failure of a RSS can be done using both Limit Equilibrium Method software and FEM commercial software. The two design approaches gave minor safety factor discrepancies (lower than 5%). In case the structure post-construction settlements are a concern, the use of FEM software is mandatory. The paper provided some main indications and recommendations regarding the material inputs to be used for the design of the RSS using commercial FEM software.

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