Reconstruction of Road Embankment Failure Using Reinforced Geogrid: Revisiting the Site After 15 Years

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ABSTRACT

A case history of repair works of road embankment failure at KM39, Route 68, Kuala Lumpur – Bentong Road in the state of Selangor, Malaysia and an observation on the condition of the site after 15 years and some tensile strength testing carried out on buried and exposed geogrid was presented in this paper. Due to heavy rainfall in 1995, about half of the road embankment which was located at the alluvial fan of an upslope stream valley was washout. For the reconstruction of the alignment, the existing half of the alignment was converted into a buffer zone with a row of gabion as a barrier to trap any fallen debris from the upslope stream valley. A geogrid reinforced embankment was constructed to provide an additional area for the new alignment of the road.

INTRODUCTION

Debris flow occurrence on 30th June 1995 caused the failure of road embankment at KM39, Route 68, Kuala Lumpur – Bentong road in the state of Selangor, Malaysia. The Kuala Lumpur – Bentong road designated as Route 68 was a major inland road linking the state of Selangor and Pahang before the operation of Kuala Lumpur – Karak Highway in the middle of '80s. Figure 1 shows the alignment of the Route 68 as well as the failed location. Route 68 now remains as an alternative route for motorist going to Pahang via Genting Sempah as well as serving the local communities around the area.

The road embankment failure was mainly due to the wash down of materials from upslope section in form of debris flow which eroded the road embankment. The debris flows brings down eroded materials and blocked the road resulting in closure of the road to traffic. The falling debris also blocked roadside earth drain as well as damaged the guardrails. The average width of the V-shaped upslope stream valley was about 2m with gradient ranged from 25^0 to 40^0 (RAS, 1996). The stream started from almost at the ridge level of the hill and Fig. 2 shows the profile of the stream while Fig. 3 shows the sketches of failure area both at the upslope stream valley and down slope embankment. The stream valley is dry during dry season but has water during wet season. It becomes like a stream with water and debris materials flowing down and deposited on road as well as causing erosion at down slope embankment.

Fig. 1. Alignment of Route 68 and location of the failure

There were presence of a lot of loose gravels and boulders in the upslope stream valley and the stream bank. Generally, these boulders were angular in shape indicating that the source was quite near. The average size of the boulders was 0.5m in diameter (RAS, 1996).

The down slope has also been scoured by surface runoff. The scouring has over steepened the down slope gradient to about

37^0. Materials at the down slope consisted of spoils brought down from the upslope and previous fill materials, which were loosely tipped during the construction of the road. This resulted in erosion failure on the embankment face. Seepage water was also noticed on the surface of the embankment. The lateral extend of the scoured face was estimated to be about 40m and the affected height was approximately 15m below the road level.

The stream valley has some problematic geological features such as the bedding system occurring parallel to the stream slope. It has a thin soil cover, generally 1m to 2m thick and these soils are generally susceptible to surface infiltration from rainfall as well as erosion by surface runoff. The weak combination of rocks and soil also significantly contributed to the surface runoff and subsurface flow. These elements act as the triggering mechanisms to further destabilized the unstable formation which consisted of thin soil layering on rock surface, steep natural slope, high porosity of soil cover and sharp contact between soil and rock interface.

Scouring of down slope embankment was due to seepage and surface flow from upslope. Seepage flow underneath the road embankment resulted blow out of piping of road embankment materials. This initiated the scouring of the embankment surface. Flow of surface runoff over the road downwards, causing further scouring of the down slope embankment surface (RAS, 1996).

At the location of failure, the road crossed stream valley, where surface runoff from contributing catchments flow downstream. Slide masses or eroded materials from banks which are deposited on the valley floors can lead to potential debris flow thrown out onto the road.

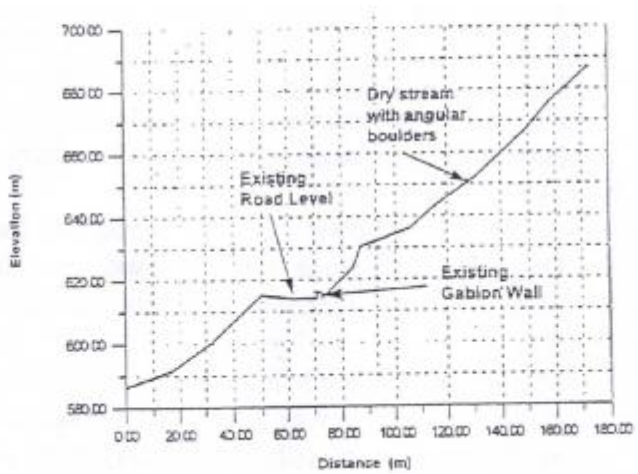

Fig. 2. Cross section profile at km 39

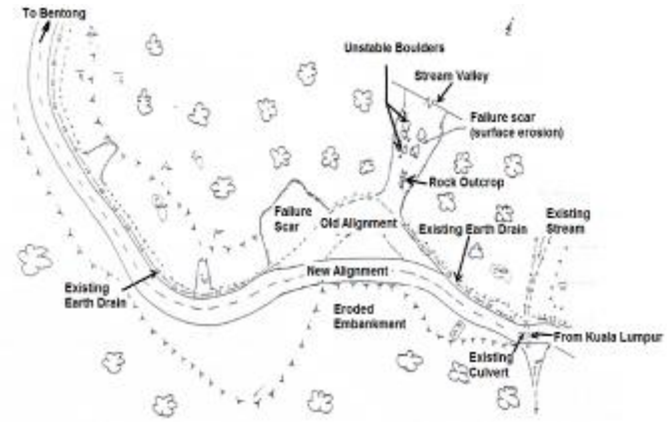

Fig. 3. Sketches of failure area

If debris flow occurs, it can block the road and may hit passing vehicles. To protect road user from possibilities of debris flow impact, a row of gabion wall as a barrier was constructed at the mouth of the valley before it reach the road alignment. The failed road embankment was repaired by reconstructing the embankment with georid reinforcing system. Reconstructing works of the reinforce embankment and observation after 16 years will be fully described in detail and presented in this paper.

SITE INVESTIGATION

Site investigation works that were carried out on the failed area included 5 boreholes and 26 numbers of Mackintosh probes for the purpose of mapping the subsurface soil profile of the area. Figure 4 shows the location of the boreholes and Mackintosh probes. The probes were carried out along 3 parallel lines. Boreholes 4, 5 and 8 were carried out along the lowest line while borehole 6 was carried out at the top most line. All the probes as well as boreholes 4, 5, 6 and 8 were performed in the failure zone. Borehole 7 was done at the toe of the stream valley.

Figure 5 shows the cross section of the slope which include the proposed geogrid reinforced embankment and the soil profile at BH 5, 6 and 7. Typical results of the Mackintosh probes MP 5, 10 and 19 are also shown in Fig. 6. It was found that the subsoil profile consists of soft and hard clay layer of about 6m depth followed by medium dense sand and weathered Sandstone later. The depth of the weathered sandstone layer was located between 2m (BH7) and 10.6m (BH6) below the existing ground. The base of the geogrid embankment was designed at a depth of 3.5m below the existing ground level at the location where BH5 was carried out. From BH5, the base of the geogrid embankment was placed on stiff sandy silty Clay with SPT 'N' value of 8 blows/ft. sandstone was found at about 3m below the proposed base level. Based on the results of the Mackintosh probes MP5, 10 and 19, the compressible/loose soil (<20 blows/ft) was about 3m below the existing ground level.

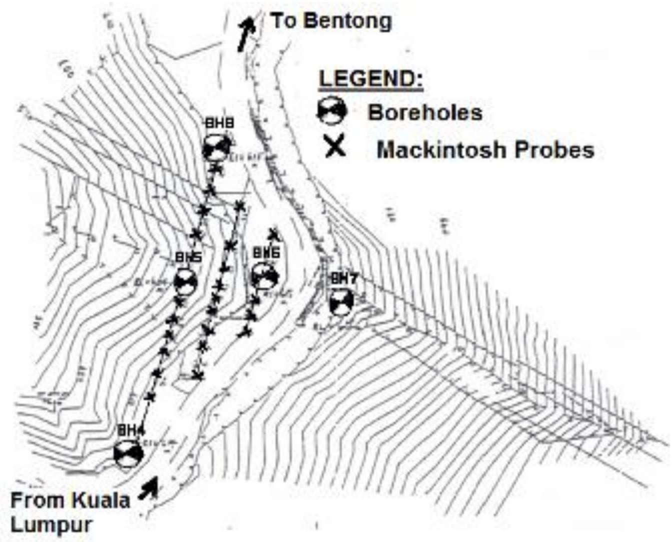

Fig. 4. Location of borehole and Mackintosh probes

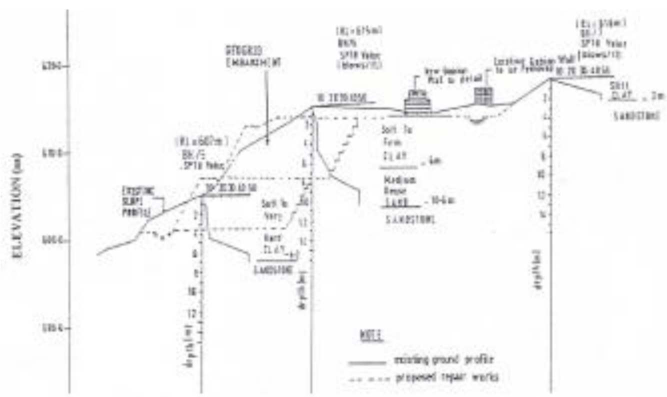

Fig. 5. Cross section of slope and subsoil profile

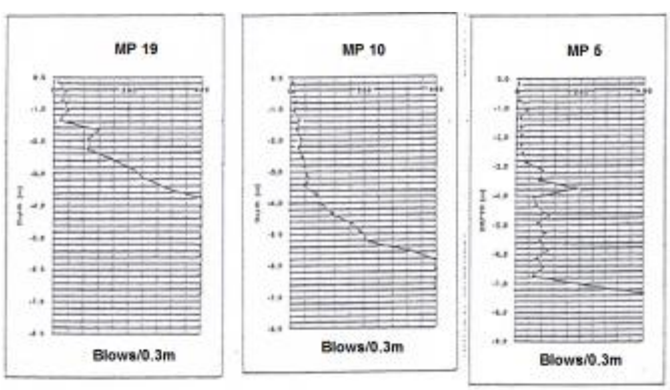

Fig. 6. Typical results of Macintosh probes at km 39

REPAIR WORKS

The repaired road was realigned further away from the toe of the upslope stream valley. A buffer zone to trap debris was constructed at the toe of the upslope using a row of gabion wall. Geogrid reinforced system was recommended due to the presence of a gentler downslope gradient for placing the base of the geogrid reinforced wall safely. In addition, the geogrid reinforced system was preferred because the embankment can be constructed to a much steeper batter thus increasing the space for the road alignment works (Jewell, 1982). Roadside drain replacing the existing earth drain was constructed to enhance the channelized surface flow on the road to the proper discharge point.

Based on the site investigation results and the topography of the area, the base level of the geogrid embankment was constructed at the reduced level of 601m. Slope stability analysis using soil parameters obtained from soil investigation works were carried out as shown in Figure 7 and 8. Figure 7 shows the slope profile and soil properties used as design parameters while Fig. 8 shows the slope stability analysis. It was found that the factor of safety is 1.26 which is greater than minimum FOS of 1.25. Figure 9 shows the plan of repair works. The details of the geogrid reinforced embankment are shown in Fig. 10. The height of the embankment was 12m with a 2m wide berm at the mid-slope while the width was 13m. The gradient of the embankment was 2:1.

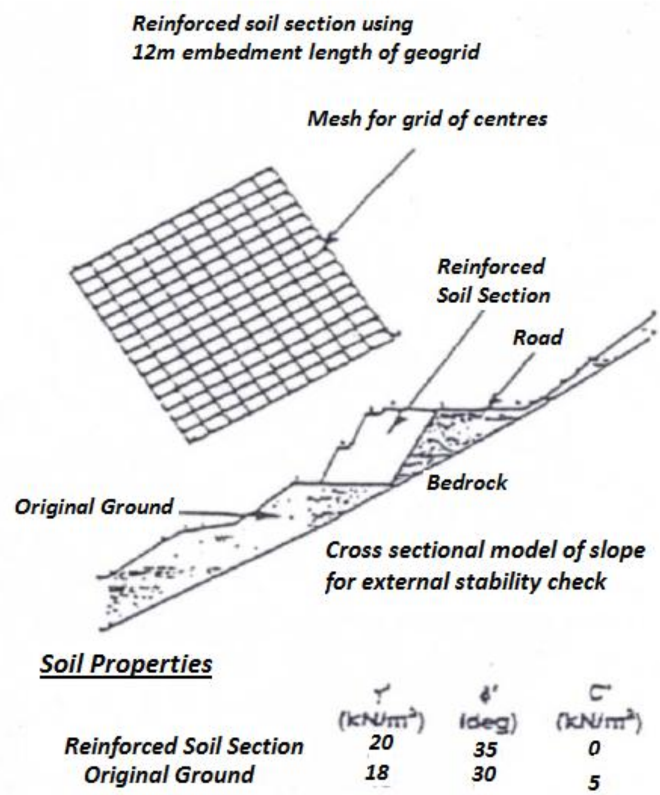

Fig. 7. Slope profile with geogrid wall and soil properties

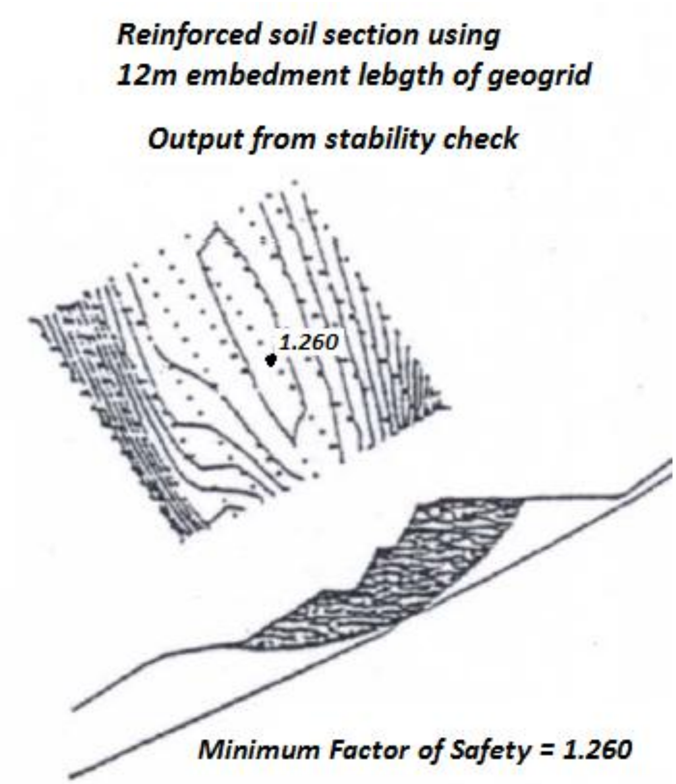

Fig. 8. Slope stability analysis

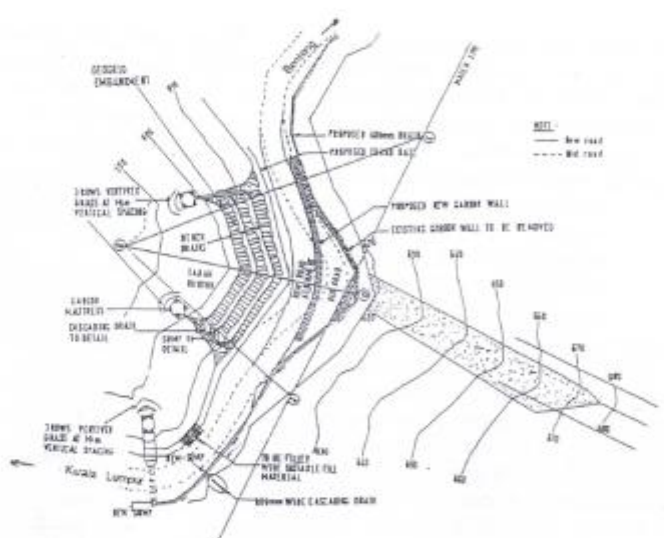

Fig. 9. Plan view of repair works

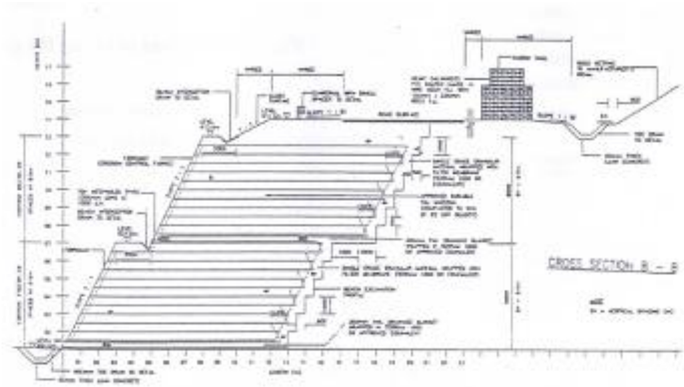

Fig. 10. Detail of geogrid reinforced embankment

Drainage was provided at toe, berm and top of the geogrid reinforced embankment and they were connected to cascading drain at both ends of the embankment. A gabion matress of 5m long by 2m wide was constructed at the base together with 3 rows of Vertiver grass with vertical spacing of 1m to prevent toe erosion. Due to the presence of the stream valley/slope hollow at the upslope section, an underground water path was detected underneath the failed embankment. In order to allow this under groundwater to flow out of the geogrid reinforced embankment, 300m thick drainage blankets wrapped in filter geotextile Terram 1000 were provided almost horizontally (about 2% gradient) at the base and at berm level of the embankment. The drainage blanket at the base was connected straight to the toe drain. Weep holes having diameter of 75mm by 1200mm long with spacing of 1000mm were installed to drain seepage water at berm level since the berm drainage blanket was stopped about 500mm inside the embankment. Another drainage blanket was constructed between the end of the geogrid embankment and the benching of the existing slope. It consisted of single graded granular material wrapped with filter geotextile Terram 1000 with an effective width of about 1m to prevent surface erosion at the face of the geogrid embankment slope, fibromat was installed and later was covered with hydroseeding.

The length of the geogrid inclusion was 12m and equally spaced at 0.5m vertically. Selected residual soil was used as the backfill materials. Geotextile Fortract 110/30-20 was placed in the first berm and first half of the second berm while a slightly lower strength geotextiles Fortract 80/30-20 were placed in the second half of the second berm of the embankment. The weak and failed soil slope was removed and benching was constructed on firm ground. The height of the benching was 1m by 0.5m wide. The direction laying of the geogrid and sequence of construction for the geogrid embankment is shown in Fig. 11 and 12 respectively.

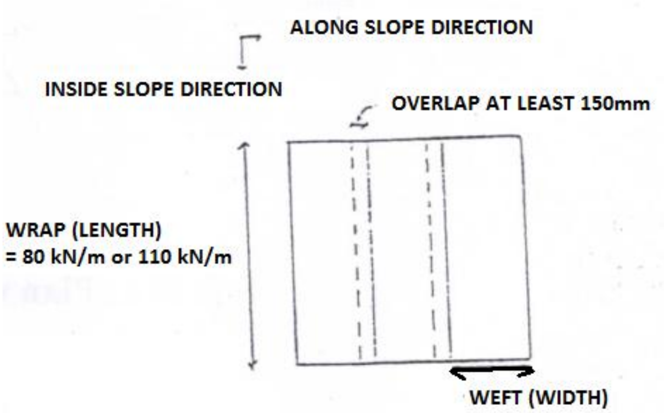

Fig. 11. Direction of laying the geogrid

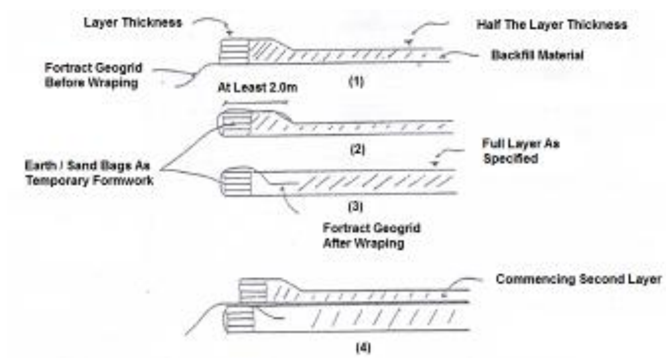

Fig. 12. Construction sequence of geogrid wall

The sequences of construction for the geogrid embankment are as follows:

a) The geogrid was unrolled on the drainage blanket and run over the temporary formwork. Four layers of sandbags were stacked on the geogrid and were used as the temporary formwork to allow for compaction of the fill (Fig. 12 (1)). Care was taken so that wrap direction of the geogrid was laid inside the slope.
b) Dumping of the fill material and compaction was carried out at every half-layer thickness (every 250mm thick). However, the front of the fill was made to the full layer thickness. Upon reaching 95% dry density compaction, the geogrid was folded over the compacted surface and the wrap around the surface was 200mm to allow for facing anchorage (Fig. 12 (2)).
c) The complete placement and compaction of the fill for the first layer was done immediately and sandbags for the second layer were made ready (Fig. 12 (3)).
d) The procedure was repeated until the desired crest level was reached (Fig. 12 (4)) over the compacted surface and the wrap around surface was 2000mm to allow for facing anchorage (Fig. 12 (2)).
e) The directions of laying the geotextile and the minimum lap length of the geotextile are shown in Fig. 11.
f) Note on compaction:

   i. All construction plants and other vehicles having mass exceeding 1000kg were kept at least 2.0m away from the back of the facing,
   ii. Within 1.5m of the back of the facing, the plants used for compacting of the fill material were restricted to vibro roller having mass per meter width of roll not exceeding 1300kg with a mass not exceeding 1000kg.

The cost of repair works was about RM1.67million. It was repaired on a design and construct basis and the rates were based on negotiation between the government and the contractor. The contract period was 275 days. The date of site possession was on 1st December 1996 and was completed on 31st August 1997.

Figure 13, 14 and 15 is a series of photo taken on various times shows progress of vegetation growth on the slope surface. Within four years construction completed, fast growing vegetation covers transformed the constructed slope surface into thick bushes as shown in Fig. 15.

There is no evidence of any crack or settlement on the road surface or at the top of the geogrid embankment on the last visit to the site in 2001.

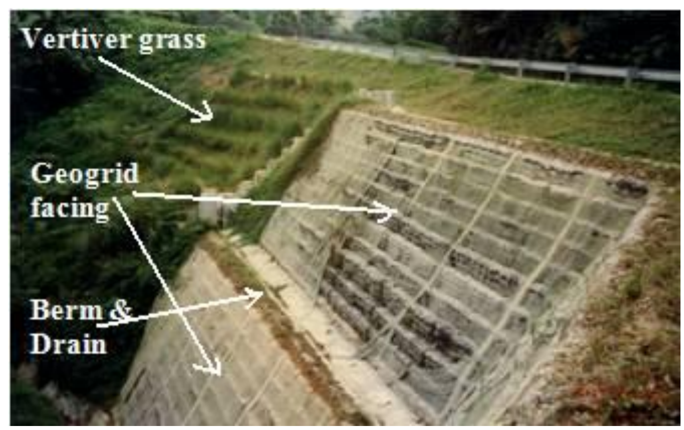

Fig. 13. Photo taken on May 25, 1998

Fig. 14. Photo taken in 1999

Fig. 15. Photo taken in 2001

REVISITING THE SITE AFTER 15 YEARS

On September 11, 2012, authors have visited the sites for inspection and to obtain geogrid sample for tensile laboratory testing. Observation from the visit and result of tensile laboratory testing was presented here.

<u>Site Observation</u>

Inspection carried out on the front side of gabion wall (facing upstream) found that there is no evidence of debris flow occurs after it's constructed or may be the trapped debris has been removed by maintenance team.

Fig. 16. Longitudinal crack was observed parallel to the edge line of the road

There is single longitudinal crack observed parallel to the edge line of the road on the reconstructed side of the embankment (Fig. 16). The crack is relatively small and if the time after construction was considered (15 years), it was acceptable. The crack is minor and maybe just shallow crack on the road pavement only.

After 15 years, the vegetation or bushes covering the slope surface are more mature as shown in Fig. 17 compared to condition 11 years before as shown in the photo in Fig. 15 which was captured in 2001. This is typical beginning of the tropical secondary forest in Malaysia.

Fig. 17. Photo taken on September 11, 2012 shows the site was covered with thick vegetation

Fig.18. Photo at first berm taken on September 11, 2012 shows the site was covered with thick vegetation

Down to the first berm from top (Fig. 18), the slope surface was covered by 3 to 5m thick bushes. The berm drain was intact, still functioning with no sign of cracking along the berm. Layers of full layer thickness (500mm) of geogrid facing were intact, covered by thin layer of fern and shrubs (Fig. 19). The layers of sandbags used as geogrid facing or temporary formwork during construction were also intact, in the original form as shown in Fig. 20. As usual, due to wet or moist condition at the floor of most tropical forest, especially at hilly area, the geogrid lines were become a medium of algae growth.

Walkthrough observation along the toe of embankment (boundary or construction site and primary forest) shows there are no sign of crack, failure or any distress.

Fig.19. Photo taken on September 11, 2012 shows layers of full layer thickness (500mm) of geogrid facing

Fig. 20. Photo taken on September 11, 2012 shows geogrid facing clearly intact 15 years after it was constructed

<u>Sampling and Laboratory Testing</u>

To determine the deterioration of strength of geogrid after 15 years it was constructed, laboratory tensile testing was carried out on the geogrid sample taken from the site. Two groups of sample were taken: buried geogrid and exposed geogrid. Buried geogrid will present the geogrid in constant loading with protection from ultraviolet and other external factor while exposed geogrid will present the geogrid in constant loading with no protection.

Three samples were taken from the uppermost berm where Fortract 80/30-20 geogrid were used to represent buried geogrid and it was named as BR1, BR2 and BR3. Four samples were taken at the toe of second berm (6m below uppermost berm) where Fortract 110/30-20 geogrid were used to represent exposed geogrid, was named as BR4, BR5, BR6 and BR7.

Figure 21 shows sampling of buried geogrid from top layer of geogrid embankment where the soil covering the geogrid was removed before a piece of 600mm (Wrap) by 200mm (Weft) was cut. Figure 22 shows sampling of exposed geogrid from the facing was in the process. The samples were then washed because it should be freed from soil, sand and algae before sending it to the laboratory. Figure 23 shows the cleaned samples of geogrid with minimum size of 600mm (Wrap) by 200mm (Weft) ready to be sending to an accredited laboratory for tensile testing.

Fig.21. Sampling of buried geogrid from top layer of geogrid embankment

Fig. 22. Sampling of exposed geogrid from the facing

Due to the Fortrac 80/30-20 and 110/30-20 geogrid was high strength geogrid, the Wide Width Tensile Test was carried out using split roller grips on the single strand of geogrid as shown in Fig. 24. This test procedure is based on ISO 10319-2008. A strain rate of 20% per minute was applied. The results were then multiply by 40 to get kN/M unit.

Fig.23. Samples of geogrid with minimum size of 600mm (Wrap) by 200mm (Weft) ready to send to an accredited laboratory

Fig. 24. Wide Width Tensile Test using split roller grips based on ISO 10319-2008

Table 1. Results of Wide Width Tensile Test (based on ISO 10319-2008)

Sample No.	Location of Geogrid Samples Obtained	Type of Geogrid Installed	Strength Tested After 15 Years (kN/m)	% Reduction
BR1	Top (buried)	Fortrac 80/30-20	78.80	1.5
BR2			75.27	5.9
BR3			79.62	0.5
BR4	Toe of Berm No. 2 (exposed)	Fortrac 110/30-20	104.47	5.0
BR5			107.47	2.3
BR6			92.61	15.8
BR7			90.58	17.7

Table 1 shows the results of Wide Width Tensile Test (based on ISO 10319-2008) carried out on all seven samples. For buried Fortrac 80/30-20, constant percentage reduction of strength between 0.5% and 5.9% with an average of 2.63% was observed. For exposed Fortrac 110/30-20, the percentages reduction of strength is between 2.3% and 17.7% with an average of 10.2% and standard deviation of 7.68%. Glaring in difference of strength reductions is difficult to explain. Is it the contractor tries to cut cost by using lower grade of geogrid? Noted that the only available grades of Fortrac geogrid attached in the Design Report (RAS, 1996) are 110/30, 80/30, 55/30, 35/20 and 20/13.

If we ignore the last two samples (BR6 and BR7), an average percentage reduction of strength for both condition is not much difference. This means that Malaysian tropical environment effect of ultraviolet, temperature and humidity) didn't much effect on the strength of geogrid.

Figure 25 shows a graph of time to rupture, under varying load, for Diolen yarn, as suggested in the Roads and Bridges Agrement Certificate No 92/69 (BBA, 1992). From this graph the characteristic strength (P_{char}) above which the material will fail in tension, can be determined for a given design life. If we put the average percentage of strength for both condition in this study into the graph, it was much higher than the suggested strength after 15 years it was installed. This high percentage of geogrid strength is may be due to the geogrid are not exposed to constant loading based on the location of sample taken: at the top most geogrid layer (thin overburden) and at the facing of slope.

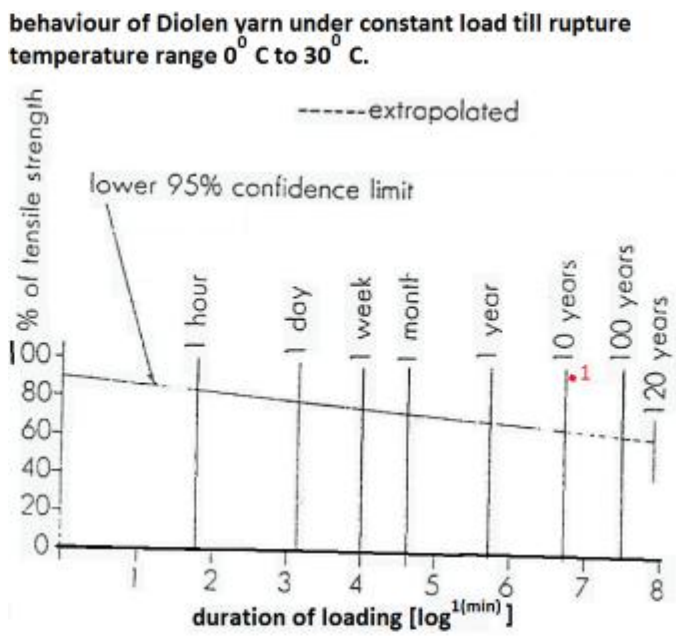

Fig.25. Time to rupture (BBA, 1992)

SUMMARY AND CONCLUSION

A case history of repair works of road embankment failure at km 39 Route 68, Kuala Lumpur – Bentong Road in the state of Selangor, Malaysia and an observation on the condition of the site after 15 years and some tensile strength testing carried out on buried and exposed geogrid was presented in this paper.

About half of the road embankment which was located at the alluvial fan of an upslope stream valley was washout after heavy rain in 1995. For the reconstruction of the alignment, the existing half of the alignment was converted into a buffer zone with a row of gabion as a barrier to trap any fallen debris from the upslope stream valley. A geogrid reinforced embankment was constructed to provide an additional area for the new alignment of the road.

Based on the site investigation results and the topography of the area, the base level of the geogrid embankment was constructed at the reduced level of 601m. Slope stability analysis using soil parameters obtained from soil investigation works were carried out. It was found that the factor of safety is 1.26 which is greater than minimum FOS of 1.25. The height of the embankment was 12m, separated into two batters with a 2m wide berm at the mid-slope while the width was 13m. The gradient of the embankment was 2:1.

The length of the geogrid inclusion was 12m and equally spaced at 0.5m vertically. Selected residual soil was used as the backfill materials. Geotextile Fortract 110/30-20 was placed in the first berm and first half of the second berm while a slightly lower strength geotextiles Fortract 80/30-20 were placed in the second half of the second berm of the embankment. The weak and failed soil slope was removed and benching was constructed on firm ground. The height of the benching was 1m by 0.5m wide.

On September 11, 2012, authors have visited the sites for inspection and to obtain geogrid sample for tensile laboratory testing. Inspection carried out on the front side of gabion wall (facing upstream) found that there is no evidence of debris flow occurs after it's constructed. Minor longitudinal crack observed parallel to the edge line of the road on the reconstructed side of the embankment. Vegetation or bushes covering the slope surface are more mature compared to condition on the last site visit eleven years ago. This is typical beginning of the tropical secondary forest in Malaysia. Walkthrough observation along the toe of embankment (boundary or construction site and primary forest) shows there are no sign of crack, failure or any distress.

To determine the deterioration of strength of geogrid after 15 years it was constructed, laboratory tensile testing was carried out on the geogrid sample taken from the site. Two groups of sample were taken: buried geogrid and exposed geogrid. Buried geogrid will present the geogrid with protection from harsh weather such as ultraviolet effect and other external factors, whereas exposed geogrid will present the geogrid with no protection. The size of each sample is 600mm (Wrap) by 200mm (Weft). The Wide Width Tensile Test was carried out using split roller grips on the single strand of geogrid. This test procedure is based on ISO 10319-2008.

Results of the laboratory tensile testing shows there are no significant reduction of strength observed after installation 15 years ago. It's only reduced (average) 2.63% and 3.65% for buried and exposed geogrid respectively from its original strength (if we ignore samples BR6 and BR7). It's understandable that this relatively small strength reduction is may be because they are not exposed to constant loading based on the location of sample taken: at the top most geogrid layer (thin overburden) and at the facing of slope.

The results also show that Malaysian tropical environment effect of ultraviolet, temperature and humidity) didn't much effect on the strength of geogrid.

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