Ecological Mitigations Rain-Triggered Landslides Using Construction Methods with Geosynthetics

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Abstract

Landslide is the natural disaster that affected the society in many ways. Landslide triggering by rain infiltration have occurred in most mountainous landscapes and can cause significant damage. In Thailand, the mountainous northern and southern areas are most vulnerable to landslide hazards during rainy seasons with extreme weather conditions. Similarly, in the mountainous and rugged terrain in Northern Laos near the border with China, rain-triggered landslides have affected the road improvement project in National Road (NR 1B) financed by the World Bank. Geosynthetics have been increasingly used for many ecological applications in civil/geotechnical engineering including road and railway embankments, retaining walls, slope and erosion protection, drainage/filtration and seepage control, waste containment and lining, geocontainers and geobags, etc. For mitigations of landslides, geosynthetics are mainly utilized to provide tensile strength and added stiffness to the soil which is basically strong in compression in order to provide the required safety level. In addition, geosynthetics also provide separation, filtration and drainage functions. The mitigations of landslide using construction methods with geosynthetics have been applied successfully in these full scale loading and actual case histories.

Keywords: Rain-Triggered Landslides, Slope Stability, Reinforced Earth, Geosynthetics

1. INTRODUCTION

Landslide, as adopted by the Working Party of World Landslide Inventory, is "the movement of mass rock, earth, and debris down a slope" (Cruden, 1991). It is a common geological phenomenon in many parts of the hilly and mountainous terrains of the world. The causes of landslides have long been studied and it is considered that it would be more appropriate to discuss causal factors including both "conditions" and "processes" than "causes" alone (Popescu, 1996). Accordingly, based on their effects, landslide causal factors are categorized as 'preparatory' and 'triggering' (WP/WLI, 1994). With the implications of climate change (IPCC 2001; Liu et al, 2008), during the last two decades, rainfall has been well recognized as a triggering causal factor leading to disastrous consequences throughout the world including the Kingdom of Thailand and neighboring countries.

The 1998 landslide and flooding events which took place in Thailand after extremely intense rainfall, claimed 373 lives and the property damage were estimated to be US$ 280 million. Surat Thani and Nakorn Si Thamarat Provinces in the south were the hardest hit (Phien-wej et al, 1993). Then, in 2001 disastrous landslide and flooding events took place in Phetchabun Province in northern Thailand. The event claimed 136 lives with over 5 million US Dollars in property damages (Yumuang, 2006). The aforementioned events and the subsequent landslide events which occurred in Tak and Mae Hong Son Provinces in 2004, respectively, and those that occurred in Uttaradit, Phrae and Sukhothai Provinces in May, 2006 (Tantiwanit, 2007, ADPC, 2006) were among other landslide events which has been on the increase and causing damages, particularly, to the road network in the hilly and mountainous terrains during the last two decades.

The Geotechnical Engineering Research and Development Center (GERD), Kasetsart University, Thailand has developed landslide database of Thailand which contains of almost 40 years of information on landslide events starting from 1970. From the database it was found that there are 2 types of landslide which can be classified based on the extensive of losses, namely: limited area landslide and large area landslide. Figure 1 shows the characteristic of each type of slide. It was found that more than 95% of limited area landslides are always

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Fig. 1 Limited and large area landslide (Soralump, 2010)

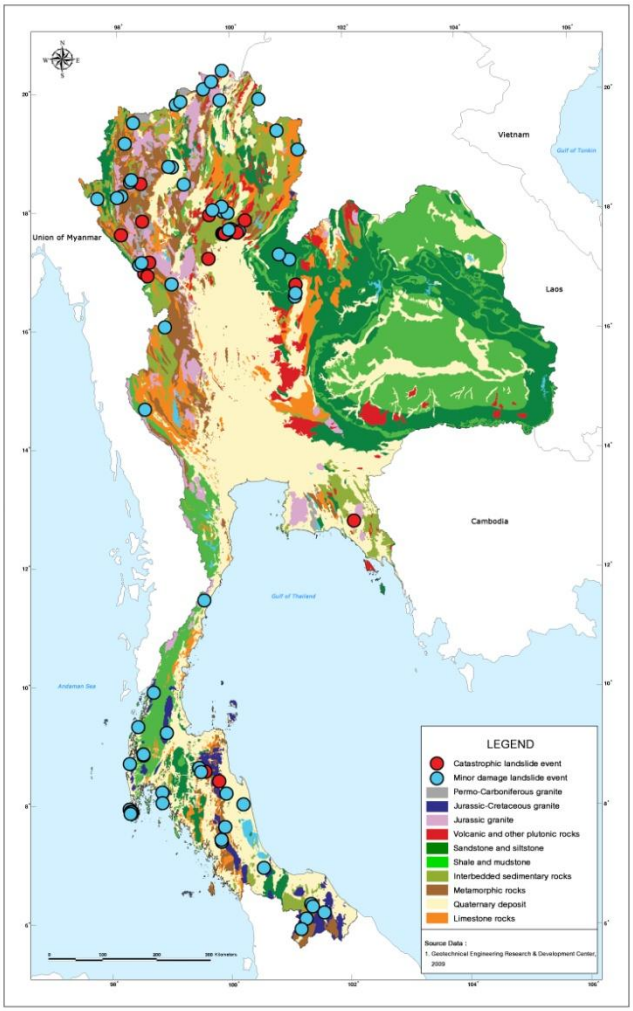

Fig. 2 Landslide events in Thailand from 1970-2006 (GERD, 2006)

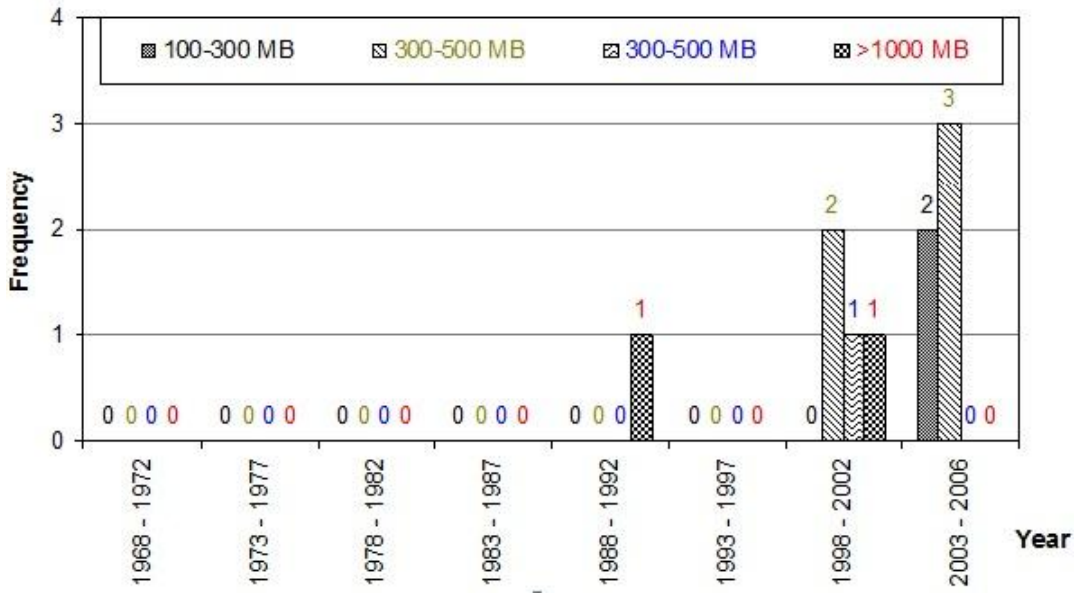

Fig. 3 Direct damage cost of greater than 100 million baht from landslide Events in Thailand (Soralump, 2007) (Exchange rate: 1 USD = 32 Bahts)

caused by the disturbance due to human activities which change the landform or surface and underground water flow characteristics. On the other hand, large area landslide is natural and mainly caused by unusual large precipitation in the area. However, there are also many evidences that deforestation or agricultural process is the main cause of large area landslide. Figure 2 shows the location of recorded landslide events from the database. It can be seen that the landslide occurred mostly in the northern and southern parts of Thailand. The frequency of the landslide event is increasing sharply during the last decade starting from 1996. The assumptions for the cause of increasing number of landslides for the past decade are: 1) Landslides naturally occur more often which might be related to the climate change (2) Mismanagement of land use due to the increasing number of population and the needs of land for producing agricultural products, that force people to stay in the landslide hazard areas (3) Combination of first and second reasons. Figure 3 shows the statistic data of landslide events that caused economic loss of greater than 100 million Bahts (exchange rate: 1USD = 32 Bahts).

It is very important to understand the behavior of residual soil or weathered rock in order to understand the landslide occurrence. Therefore, the classification of rock group that specifically related to landslide is needed at the first place. Similar type of rocks, based on their formation, age and statistical record of landslide, have been grouped together by Soralump et al. (2007) resulting in 10 rock groups which are: Group 1: Carboniferous-Permian Granite has area of 0.74%, Group 2: Jurassic-Cretaceous Granite has area of 1.84%, Group 3: Jurassic Granite has area of 4.55%, Group 4: Volcanic rock and other intrusive rock such as Basalt, Andesine, Diorite etc. has area of 3.04%, Group 5: Sedimentary rock mainly Sandstone has area of 17.55%, Group 6: Sedimentary rock mainly Shale and Mudstone (18.79%), Group 7: Sedimentary rock, combination and interbeded or intercalated (7.17%), Group 8: Metamorphic rock has area of 6.3%, Group 9: Quarternary sediment has area of 32.89%, and Group 10: Carbonate rock mainly Limestone has area of 7.13%.

Polymeric materials providing tensile resistance and stability to soils that have low tensile strength are probably the best example of the recent day developments in geotechnical construction. Referred to as Geosynthetics, these materials are defined as planar products manufactured from polymeric materials (synthetic) used with soil, rock, or other geotechnical related material (geo) as part of a civil engineering project. There are few developments that have had such a rapid growth and strong influence on so many aspects of civil engineering practice as geosynthetics. In many cases, it has been proven that the use of a geosynthetics had significantly increased the safety factor, improved performance, and reduced costs in comparison with conventional design and construction alternatives (Koerner, 1998; Holtz et al., 1997).

Natural fibers are now utilized called limited life geosynthetics (LLGs). Artidteang et al. (2013) and Chaiyaput et al. (2014) have demonstrated successfully the application of Kenaf LLGs for short term reinforcement of embankment on soft ground. It is implied that such ecological geosynthetics can be applied to mitigate slope failure in embankment constructions on soft ground. Meanwhile, root reinforcements from vegetations and trees have been utilized (Voottipruex et al, 2008).

It must be pointed out that failures can also occur on reinforced soil structures. A case, for instance, is a 7.4 m high geosynthetic reinforced segmental retaining wall (SRW) that

collapsed after 2 months of completion during rainy season in Korea (Yoo and Jung, 2006). Subsequent investigation revealed that the inappropriate design and low quality backfill materials were mainly responsible for the collapse, although the primary triggering factor was the rainfall infiltration. In addition, Lackner et al. (2013) presented new concepts and experimental data concerning prestressed geogrid reinforcements to reduce lateral and vertical displacements.

In this presentation, the mechanics of rain-triggered landslides are discussed. Then, the full scale study comparing the polymer and metallic reinforcements are presented. Subsequently, the successful mitigations of rain-triggered landslides using geosynthetics are demonstrated based on actual case histories.

2. THE GOOD, BAD, UGLY AND BEAUTIFUL

Landslides occur due to unstable natural or artificial slopes when the shear stress ("Bad") exceed the shear strength ("Good") as indicated in Fig. 4. The factors leading to instability can generally be classified as follows:

a) Those causing increased stress:
- Increased unit weight of soil by wetting due to rainfall
- Added external or surcharge loads such as buildings or structures
- Steepened slopes either by natural erosion or excavation of the toe
- Applied cyclic loads due to earthquakes

b) Those causing reduction in soil shear strength:
- Absorption of water and subsequent of loss suction pressures
- Increased pore pressures with increased water contents
- Cyclic loads
- Loss of cementing materials
- Weathering process
- Strength loss with excessive strain of sensitive clays
- Freezing and thawing

Signs of abnormal weather and climate change ("Ugly") are evident nowadays including rain-triggered landslides, riverbank and coastal erosions, flooding and rising sea levels compound disasters involving hydraulic and geotechnical aspects. As consequent of extreme weather events, the increasing intensity of storms and typhoons yields priorities in prevention of riverbank and coastal erosions as well as risk assessment of landslides and debris flows and their mitigations.

Geosynthetics ("Beautiful") are now increasingly used for many applications in civil/geotechnical engineering including road and railway embankments, retaining walls, slope and erosion protection, drainage/filtration and seepage control, waste containment and lining, geocontainers and geobags, etc. For mitigations of landslides, the reinforcement function of

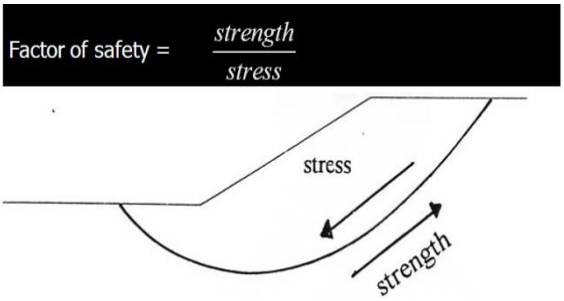

Fig. 4 Schematic illustration of factor of safety in slope stability analysis

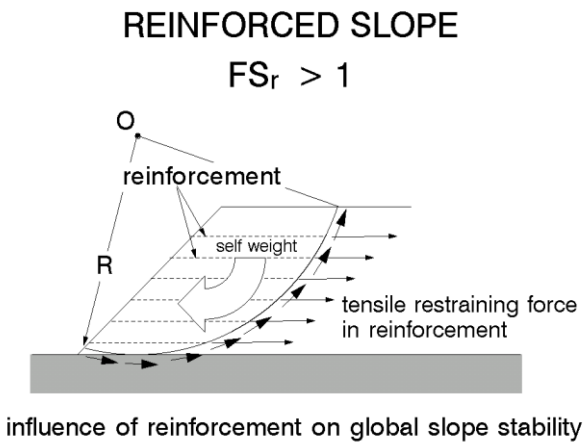

Fig. 5 Geosynthetics soil reinforcement

geosynthetics is mainly utilized to provide tensile strength and added stiffness to the soil which is basically strong in compression (see Fig. 5).

3. MECHANICS OF RAIN-TRIGGERED LANDSLIDES

Conceptual models for the hydrologic response leading to rain-triggered landslides have existed for some time. The model that is perhaps the most well recognized is that presented by Campbell (1975). This model is based on the idea that the regolith has increasing density and decreasing hydraulic conductivity with depth and if the rainfall rate exceeds the deep percolation rate, a perched water table forms in the regolith. The implicit assumption is that the flow in this saturated zone is roughly parallel to the slope, and that, in the most critical analysis the saturated zone reaches the surface and the pore water pressure is constrained by this height.

Brand (1980) discussed the mechanism of rain-triggered landslides with reference to residual material acknowledging the fact that they are heterogeneous and, under dry conditions, exists as unsaturated. It was stated that the mechanism of rain-triggered landslides is that water infiltration causes a reduction in the matric suction in the unsaturated soils. This gives rise to a reduction in the effective stress on the potential failure surface, decreasing the soil strength to a state at which equilibrium can no longer be sustained in the slope (Fig. 6).

RAINFALL (P_t mm/min)

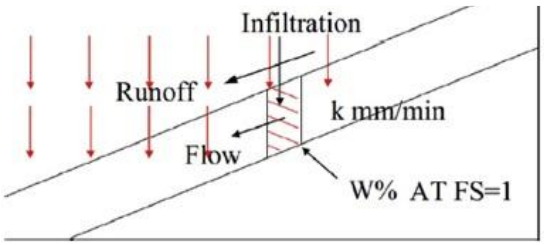

Fig. 6 The modeling of rainfall triggered slope failure (Kunsuwan, 2005)

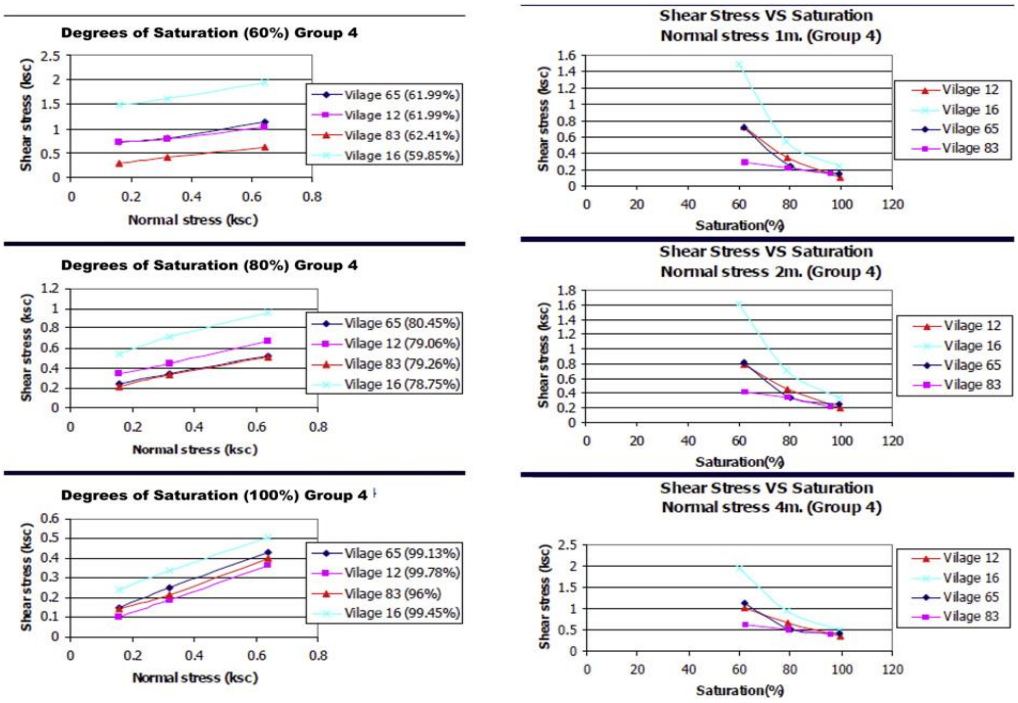

Fig. 7 Strength reduction characteristic from increasing of soil moisture content (Soralump and Torwiwat, 2009)

The key elements affects the stability of a slope that changes due to the rainfall would be: (1) Reduction in shear strength of soil due to the increase of soil moisture. (2) Permeability of residual soil in unsaturated condition and also the permeability of the underlying base rock; (3) The expected failure planes that actually controlled by the soil type, thickness and slope angle; (4) Rainfall pattern of the study area; (5) Initial saturation contour of the subsoil; and, (6) Absorption and retention characteristic of land cover. The change in shear strength of soil in unsaturated zone due to the changing in soil moisture content (or matric suction) can be determined directly in laboratory by performing the direct shear test to the soil samples with various degree of saturation. In classical way, the above characteristics can be determined based on strength equation for unsaturated soil introduced by Fredlund (1978). This method required the determination of Soil Water Characteristic Curve (SWCC) in addition to effective strength parameters. Undisturbed samples of residual soil located in each rock group have been collected. The total number of over 514 soil samples has been tested for their engineering properties and 307 undisturbed soil samples have been specially tested their shear strength reduction behavior by direct shear testing method (Soralump and Torwiwat, 2009). Figure 7 shows the example of strength reduction characteristics.

4. FEM 3D SIMULATION OF MSE WALL/ EMBANKMENT WITH METALLIC AND POLYMENIC REINFORCEMENTS

4.1 Introduction

A 6m high reinforced earth embankment was constructed and designed by Department of Highways (DOHs), Thailand near the Highway No.11 Phitsanulok-Uttaradit in Thailand (Duangkhae et al., 2013). Two types of reinforced earth embankment were constructed. In one side, the Reinforced Steep Slope (RSS) was

constructed at 70 degrees sloping face from the horizontal consisting of soil bags. In the other side, the Mechanically Stabilized Earth Wall (MSEW) was installed with vertical concrete panel as facing. The plan and section views of the test embankments are indicated in Figs. 8 and 9, respectively. The test embankment had dimensions of 18m long and 15m wide at the top. Three types of polymeric geogrids reinforcement were installed in reinforced steep slope (RSS) facing and two types of metallic reinforcement were installed in mechanically stabilized earth wall (MSEW) facing. Polymeric geogrids reinforcement consisted of polyester (PET), high density polyethylene (HDPE) and polypropylene (PP) and metallic reinforcements consisted of steel wire grid (SWG) and metallic strip (MS). The vertical spacing was kept as 0.5m and length of the reinforcement is 5m. Different monitoring instruments were installed to check the displacements, stresses, excess pore water pressures, groundwater table and strains in reinforcing material including inclinometers, settlement plates, total pressure cells, standpipe piezometers, vibrating wire strain gauges and fibre optic strain gauges (see Figs. 8 and 9). PLAXIS 3D (Version 2011) was utilized for the FEM numerical simulations of the embankment. The behaviour of a reinforced soil slope (RSS) and mechanically stabilized earth wall (MSEW) on hard foundation were observed and compared with the predictions from PLAXIS 3D software in terms of lateral and vertical deformations, strains and total pressures.

4.2 Components of the Test Embankment

The hard foundation of the test embankment consisted of the interlayering of dense to very dense sand and very stiff to hard silty clay. The backfill materials used in this embankment consisted of 50% lateritic soil mixed with 50% silty sand (by volume). Details regarding properties of backfill material is summarized in Table 1.

Two types of reinforcement, namely: polymeric and metallic reinforcements, were used in the reinforced embankment. Details regarding the five different polymeric and metallic reinforcements are tabulated in Table 2. The reinforcing materials are shown in Fig. 10. The photographs of the vertical face and sloping face with metallic and polymeric reinforcements, respectively, are shown in Fig. 11.

The vertical facing used in the MSEW portion of the embankment consisted of segmental precast concrete blocks 1.5 m by 1.5 m in size. The model parameters used in the numerical analysis for the precast concrete panels are summarised in Table 3.

4.3 3D Numerical Simulation

PLAXIS 3D (Version 2011) was utilised for the 3D FEM numerical simulations of the embankment. To minimise the effects of test embankment boundaries, the PLAXIS 3D discretisation was formulated and the boundary conditions were specified at distances of two times the length and width of the reinforced embankment in the x and y directions, respectively, as well as at a distance of four times the height of the reinforced embankment in the z direction. To carry out the finite element analysis of the embankment using PLAXIS 3D, a finite element mesh was created (Fig. 12), and the material properties of the embankment components were established (Table 4). The generation of an appropriate finite element mesh and the generation of properties and boundary conditions on an element level were automatically performed by the PLAXIS mesh generator based on the input of the geometry model. The values of interface coefficients (R_{inter}) for steel strip, steel wire grid, PET, PP, and HDPE geogrids were 0.87, 1.0, 0.79, 0.83 and 0.77, respectively.

4.4 Results and Discussions

The test embankment was constructed to 6 m height for 125 days. Subsequently, a surcharge of 20 kPa (1.2 m thick of fill) was added at the top of the embankment as shown in Fig. 11. The lateral deformations were monitored until 186 days after construction. The lateral deformation of each type of polymeric reinforcement (i.e., PET, PP and HDPE) on the RSS side and each type of metallic reinforcement (i.e., MS and SWG) on the MSEW side obtained from field measurements using inclinometers is compared with the numerical simulation results at 186 days after the end of the construction. The inclinometer readings and field measured values after 186 days for polyester (PET) and steel wire grid (SWG) are shown in Figs. 13 and 14, respectively. At the top of the embankment, larger deformation for PET (i.e. 21mm) and smaller deformation for SWG (i.e. 6mm) were observed. This is due to smaller value of stiffness of polymeric reinforcement. The 3 polymeric reinforcements had similar lateral deformations.

5. LANDSLIDE MITIGATION USING GEOSYNTHETICS (LAOS)

5.1 Weather Records

Weather has been extreme in 2013 which started early June and continued until December. Typhoon Trami brought heavy rainfall from 19-21 August 2013. The three days heavy rainfall was unusual. The monthly rainfall data in August and September from 1991 to 2013 are plotted in Figs. 15 and 16, respectively.

5.2 General Slope Stability and Subsurface Water Problems

The geology of Laos is complex and includes a wide variety of rock types, of a range of igneous, sedimentary and metamorphic origins. Many rock masses exposed in road cuttings and in the natural slopes are highly disturbed and jointed due to tectonic processes, and so are vulnerable to instability.

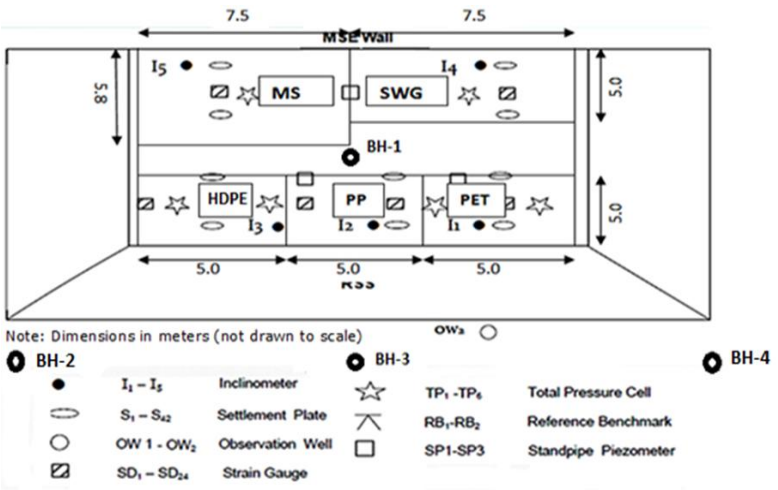

Fig. 8 Plan of MSE wall/embankment.

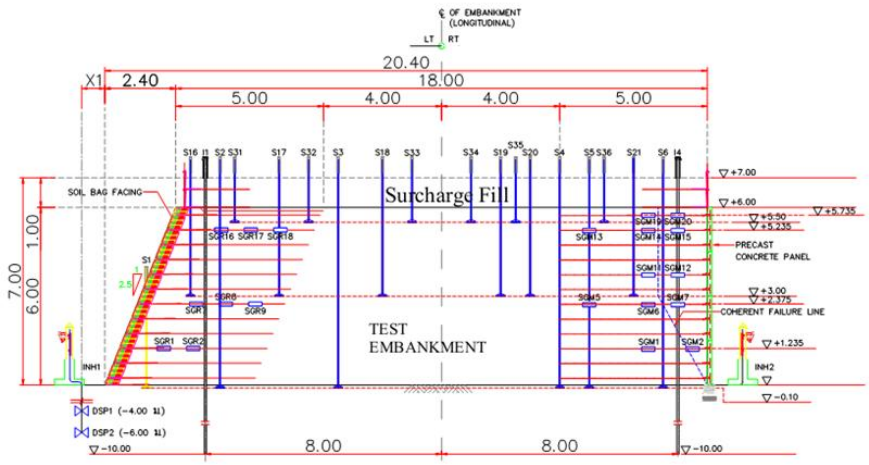

Fig. 9 Cross section of MSE wall/embankment indicating the location of the monitoring instrument.

Table 1 Properties of backfill material

Atterberg Limit Test	LL = 20.8%, PL=17.3%, PI=3.5%.	
Sieve Analysis Test	Sample No. 1 Percent finer (#200 sieve) = 0.94% Cu = 40, Cc=0.34 Sample No. 2 Percent finer (#200 sieve) = 0.14% Cu = 42.86, Cc = 0.55	
Unified Classification	Poorly graded sand (SP)	
AASHTO Classification	A-2-4(0)	
Compaction Test	Maximum dry density ($\square_{d,max}$) = 19.62 kN/m^3 Optimum water content (OMC) = 7.8 %	
California Bearing Ratio (CBR) Test	CBR = 50.5%	
Direct Shear Test	Friction angle = 42 degrees cohesion = 80 kPa	
Triaxial Test (CU test)	Test No. 1	Friction angle = 32.8 degrees cohesion = 0 kPa
	Test No. 2	Friction angle = 37 degrees cohesion = 20 kPa

Table 2 Properties of reinforcing materials

Material Name	Tensile Strength (kN/m)	Thickness (mm)	Normal Stiffness, EA (kN/m)
Metallic Strip (MS)	277.6	4.00	88,000
Steel Wire Grid (SWG)	128.1	6.00	35,000
Polyester (PET)	83.6	1.50	925
Polypropylene (PP)	91.9	1.45	1,360
High-Density Polyethylene (HDPE)	85.8	1.91	1,320

Table 3 Properties of concrete panel facing

Parameter	Name	Value	Unit
Type of behaviour	Material type	Elastic	
Normal stiffness	EA	42,000,000	kN/m
Flexural rigidity	EI	78,500	$kN.m^2/m$
Equivalent thickness	d	0.15	M
Weight	w	3.6	kN/m/m
Poisson's ratio	υ	0.15	-
Model		Plate	

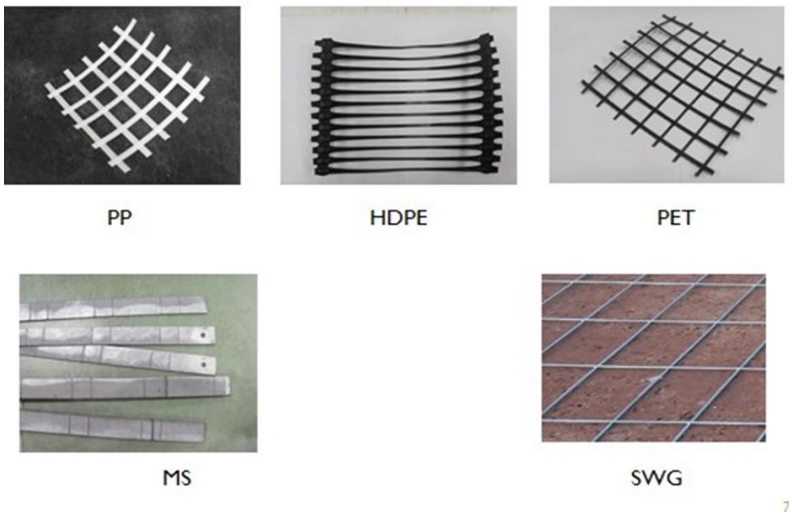

Fig. 10 Reinforcing materials

Fig. 11 Photograph showing vertical face and sloping face with metallic and polymeric reinforcements, respectively.

Table 4 Material properties

Soil Description	Model	Condition	γ_{sat} (kN/m^3)	γ_{unsat}(kN/m^3)	υ	E (kPa)	c' (kPa)	Φ' (°)	Dilation angle, Ψ(°)
Backfill	M-C	Drained	22.7	21	0.3	20,000	10	37	7
Loose clayey sand	M-C	Drained	19	17	0.3	18,000	1	33	3
Medium dense clayey sand	M-C	Drained	18	16	0.3	37,500	5	34	4
Stiff to very stiff clay	M-C	Drained	17	15	0.35	40,000	50	24	0
Very stiff clay	M-C	Undrained A	17	15	0.35	50,000	80	26	1
Hard clay	M-C	Undrained A	17.5	15.5	0.35	80,000	100	28	1

Note: Undrained A uses the effective parameters for stiffness and strength in PLAXIS 3D.

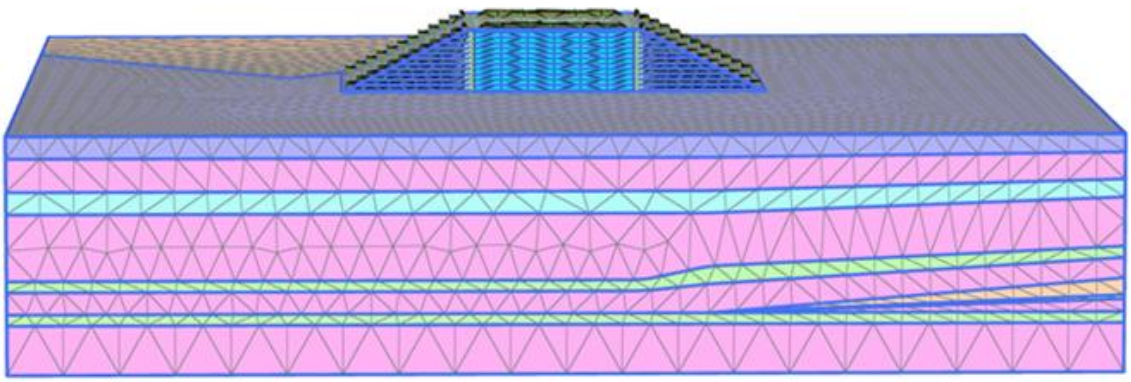

Fig. 12 3D Discretization model of MSE wall/embankment

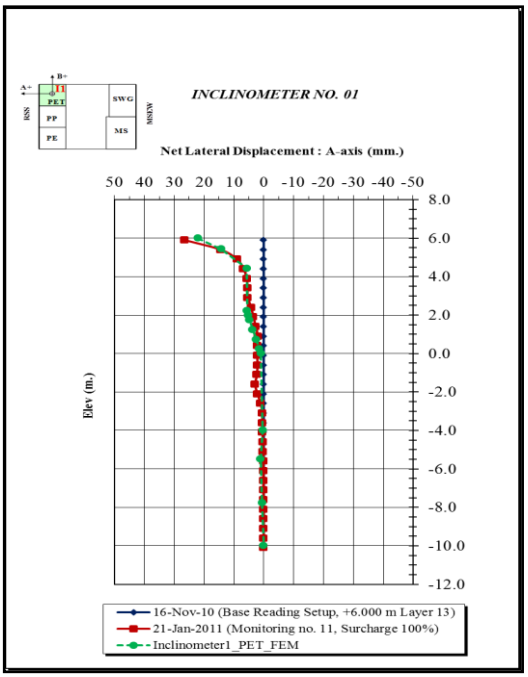

Fig. 13 Inclinometer readings at the facing of the polyester (PET) geogrid reinforcement

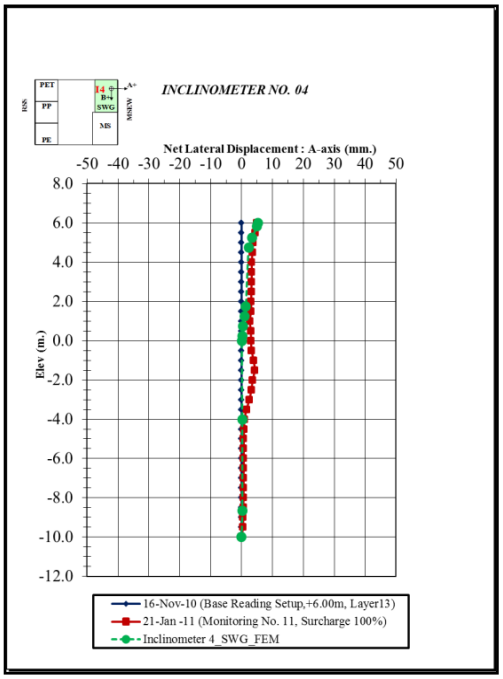

Fig. 14 Inclinometer reading at the facing of the steel wire grid (SWG) reinforcement

Lao Road Sector Project (LRSP)
Improvement of NR1B Rainfall Data (1991 - 2013)
Station: Oudomxay near NR1B

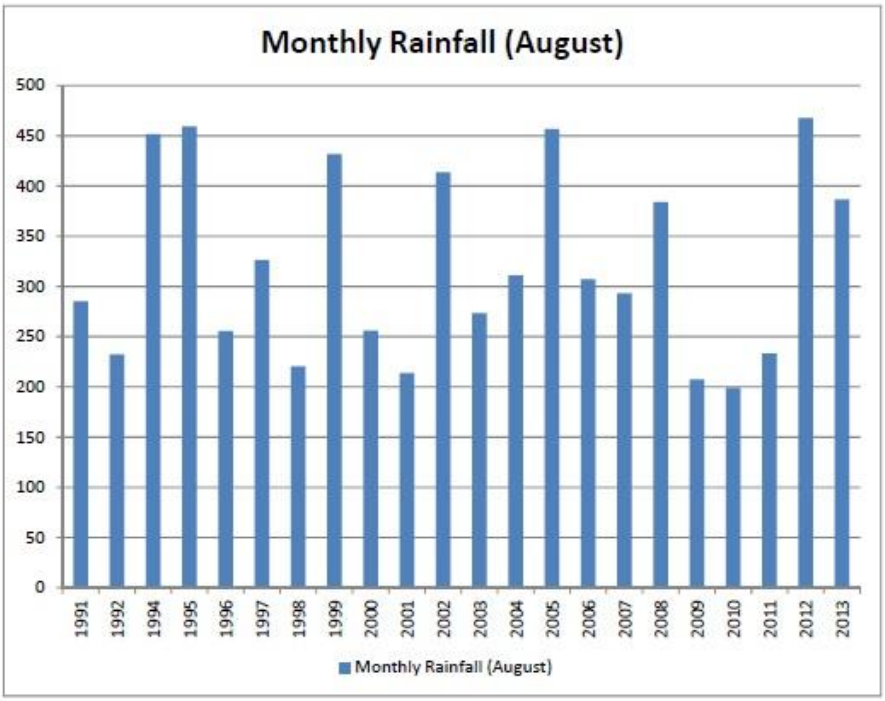

Fig. 15 Monthly rainfall for August near NR 1B, Laos

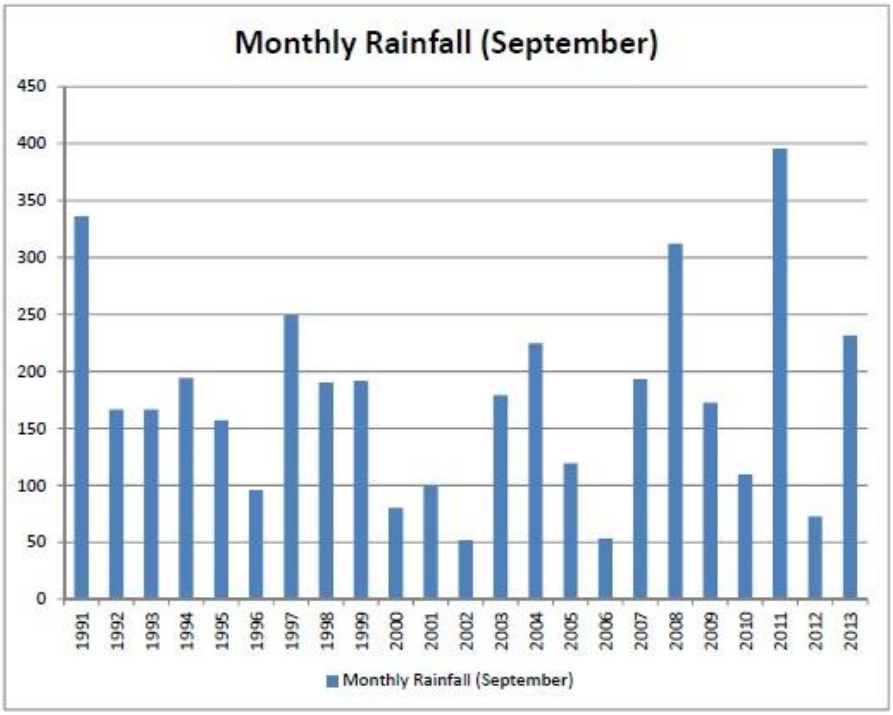

Fig. 16 Monthly rainfall for September near NR 1B, Laos

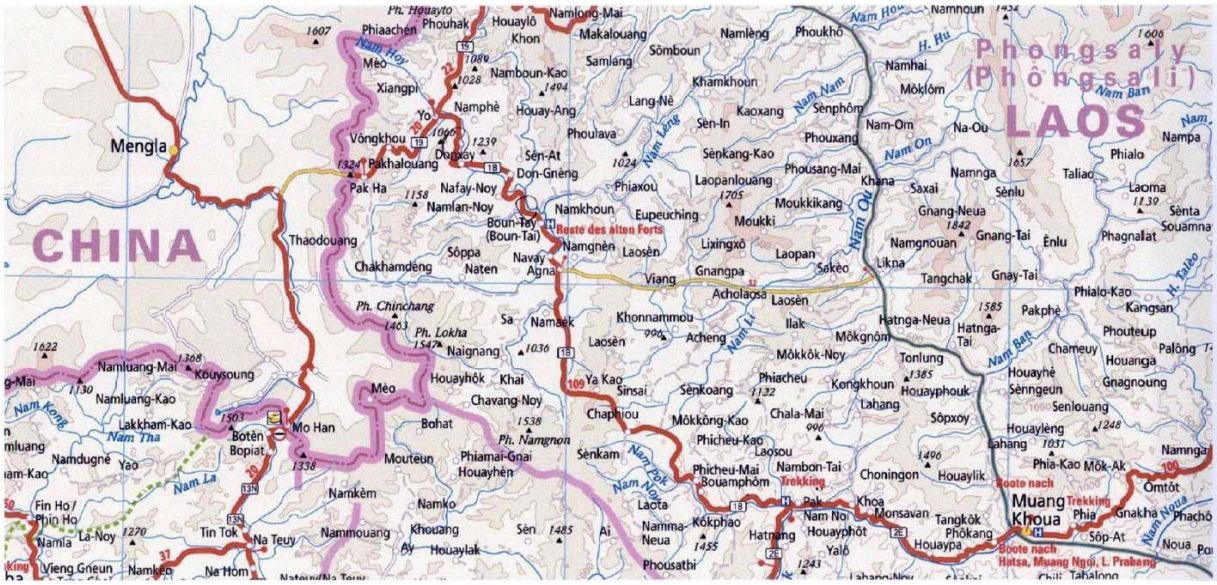

Fig. 17 Location of National Road 1B (NR1B), Laos

Furthermore, weathering under the tropical climate has led to the weakening of rock masses and has also resulted in the development of deep residual soils in places that are also prone to erosion and landslides, especially during the wet season Rainfall patterns in Laos are dominated by the south-west monsoon and the relief of the country.

Annual rainfalls of 3000mm and 4000mm are not uncommon and rainstorms can yield intense rainfall, with 100mm in 24 hours being common. This rainfall can lead to the development of high groundwater in slopes, as well as flooding in streams and rivers that give rise to erosion.

This underlying geology and hydrology, combined with the steep topography found in the vicinity of approximately 50% of the national road network, creates conditions in which landslides are common. These range from large and deep failures through to shallow and localized landslides in roadside cuttings and adjacent slopes, the latter being far the most common along the national road network.

During the heavy rains, excessive surface and subsurface water have caused slope stability problems such as sliding of cut slopes and side hill fills, and road failures such as pavement rutting and cracking.

Based on experience in Laos, during design and construction, it is impossible to locate the areas of subsurface water such as permanent or temporary water tables that can occur during heavy rains, and water springs. Usually, after they appear during the rainy season, remedial works are taken to intercept the water and take it away.

5.3 Landslides and Rockfalls

National Road 1B (NR 1B) is located near the border of Laos and China (Fig. 17). The soils in National Road 1B (NR 1B) originated from sedimentary rocks that have reached different weathering stages. They are composed of residual soils that are extremely heterogeneous, being products of weathering processes that are prone to slope failures.

The geotechnical properties, especially strength properties, are strongly affected by rainfall. The properties of the residual soil vary in depth, from those corresponding to a soft soil to those for a rock. Typical layers are difficult to identify due to gradual transition of properties in the layers. Often the upper soil layers are more porous than deeper layers. The rocks along the alignment consist mainly of shale and shist. At some few locations, weathered sandstone and slates are found.

One of the most important factors for slope stability is the water situation, especially pore pressure. A negative pore pressure (suction pressure) due to capillary forces often gives a critical slope stability. The change from a negative pore pressure to a positive pore pressure during a rainy season is the difference between a stable and unstable slope. Landslides have occurred in several locations in old landslide sections and in new locations during July to September and December 2013.

5.4 Landslides and Their Mitigations from KM 90 to KM 103

Soil erosions and slope failures occurred mainly during the widening of the road sections of National Road 1B (NR1B), Vientiane, Laos PDR (Fig. 17) along the portions passing through the soft shale deposits (KM90 to KM 103). The soft shale tends to weather into more clayey soils which is a problem soil with low shear strength and high compressibility. When wet, clayey soils losses suction pressure and generate pore pressures which decrease its shear strength. Moreover, being fine-grained soils with low permeability, clayey soils are highly erodible.

Fig. 18 Landslide at KM92

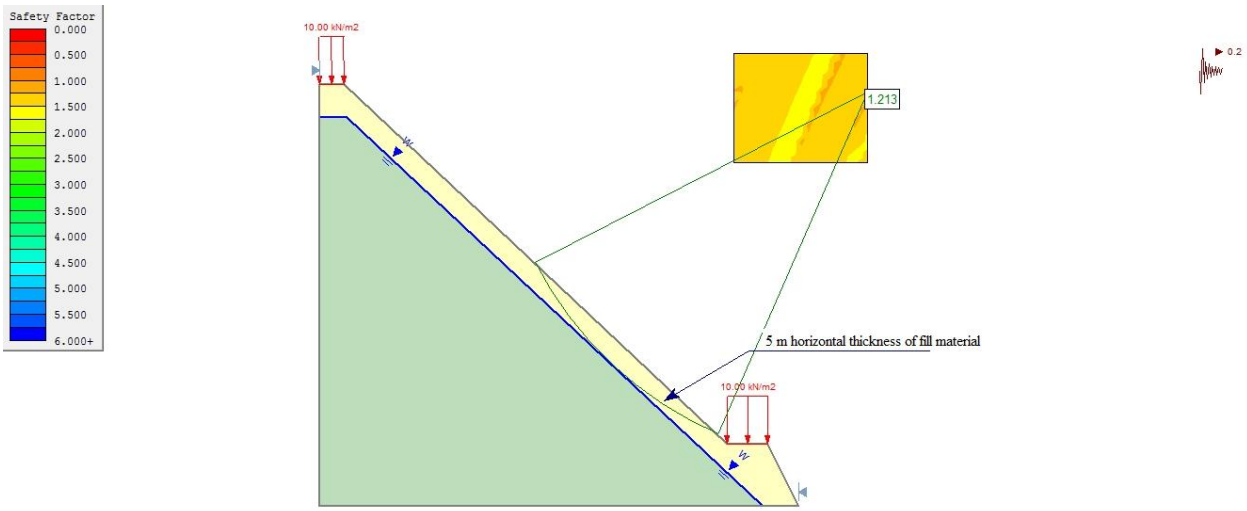

Fig. 19 External slope stability analysis at KM 92

During the widening of the road sections, additional cuts were made in the upper slopes and apparently were pushed into the lower slopes with minimum compaction and without erosion protection measures. During the rainy season, unusually high volume of rain was reported that flooded the river below the lower slopes. Consequently, the undercutting of the lower slopes occurred resulting in the slope failures of the overlying layers and eventually affected the newly widened road sections.

In Fig. 18 at KM 92, slope failures also occur in the lower slope adjacent to a river. Both external and internal slope stability were performed at the slope sections. The external slope stability analyses were done without reinforcements while the internal slope stability analyses were done with geogrid reinforcements. Results of the external slope stability are shown in Fig. 19 which indicated safe values of the factor of safety. In addition, infinite slope stability analyses with reinforcement (Tult = 150 kN/m) were done. The ultimate tensile capacity of the reinforcements was assumed to be 150 kN/m with vertical spacing of 0.5 m.

The recommended mitigation measures for this landslide is shown in Fig. 20. The lower sections of the soil erosion mitigation adjacent to the river bank consisted of gabions with 1.0 m by 1.0 m cross-section filled with large size river gravel. The gabions shall extend from 4 to 7 m in height to protect against the highest observed flood level of the river. Behind and underneath the gabions, geotextiles (200 g/m^2) were provided as separator, filter and drainage. Moreover, the

lower sections, were reinforced with polymer geogrids (*Tult* = 150 kN/m) with vertical spacing of 0.5 m in combination with the gabions. The polymer geogrids consisted of polyester (PET) geogrids.

In the upper sections of the erosion mitigation measures, a combination of 0.30 m thick mattresses filled with large size river gravel and geotextile reinforcements (*Tult* = 24 kN/m and 325 grams/square m) with 0.5 m vertical spacing shall be provided. Beneath the mattresses, geotextiles (200 g/m^2) shall be installed as separator, filter and drainage. Progress of construction and completed construction at KM 92 are shown in Figs. 21 and 22. The loose overburden layers were remarked and replaced with compacted crushed sandstone fill. The geotextile reinforcements also served as horizontal drainage.

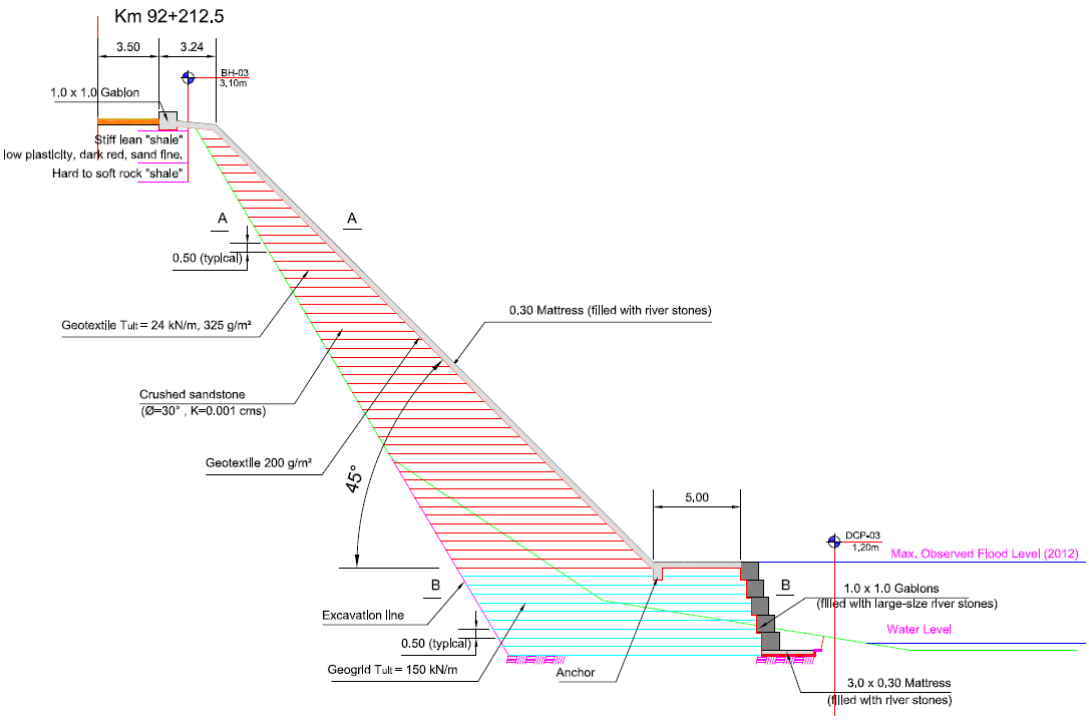

Fig. 20 Proposed mitigation using geosynthetics at KM 92

Fig. 21 Progress of construction for KM 92

Fig. 22 Completed construction at KM 92

5.5 Landslides and Their Mitigation from KM 1 to KM 75

Past design methods for roads across hillslopes emphasized achieving road with balanced earthwork i.e. equal cut and fill quantities. Thus, constructors removed materials from the hillsides (cut) and placed it along the lower sides of the road (fill) minimum compaction. During the widening of the road sections, additional cuts were made in the upper slopes and apparently were pushed into the lower slopes without adequate compaction and without erosion protection measures. The soil profile consists of dark red low plasticity silty clay derived from the weathering of shale. Moreover water seepages under the road and wetting in the lower slopes occurred with consequent slope failures and, subsequently, caused widespread cracking in the pavement of the newly widened road sections (Figs. 23a, b). These types of failures also occurred at thicker deposits of weak soil layers resulting in larger and deeper slope failures. Furthermore, some areas experienced excessive volume of underground seepages due to the presence of large catchment areas in the upper hillsides. These problems occur every rainy season and needs immediate risk assessments so that mitigation measures can be formulated in the near future.

Both external and internal slope stability were performed at the slope sections corresponding to the locations of boreholes. The internal slope stability analyses were done with high strength geotextile reinforcements. The ultimate tensile capacity of the geotextile reinforcements consisted of PEC-150 was 150 kN/m with vertical spacing of 0.5 m. The external slope stability is illustrated in Fig. 24.

The recommended mitigation measure is proposed for the slope sections as shown in Fig. 25 combination of 5 to 6m high geotextile reinforced lower slope and trench drain (1.5m deep and 0.5m wide) with TS50 (*Tult* = 15 kN/m) geotextile as separator, filter and drainage. Moreover, the lower slope shall be reinforced with 6 to 9m long PEC150 polymer geotextile (*Tult* = 150 kN/m) with vertical spacing of 0.5 m in combination with the gabion or jute bag facing. The geotextile reinforcement consisted of high strength polypropylene geotextiles. The high strength non-woven and needle-punched geotextile reinforcements can also serve as horizontal drainage.

a. Deep slope failure at KM 27

b. Deep slope failure at KM 27

Fig. 23 Deep slope failure at KM 27

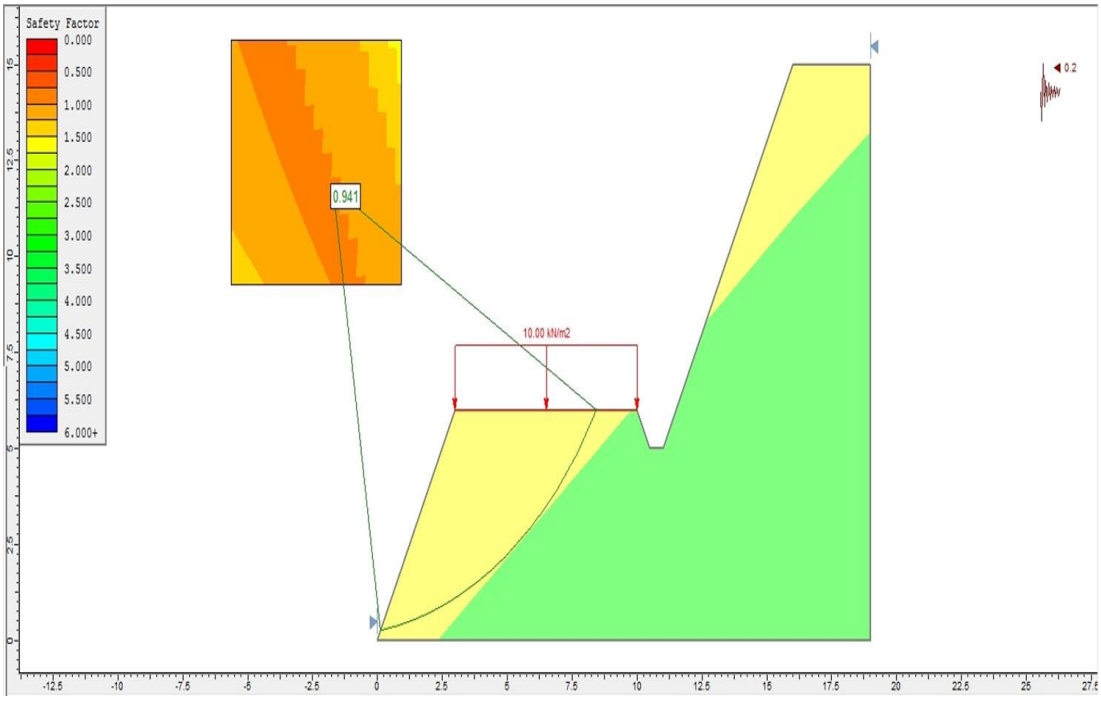

Fig. 24 External slope stability with surcharge load of 10kPa and 0.2 g of horizontal earthquake force

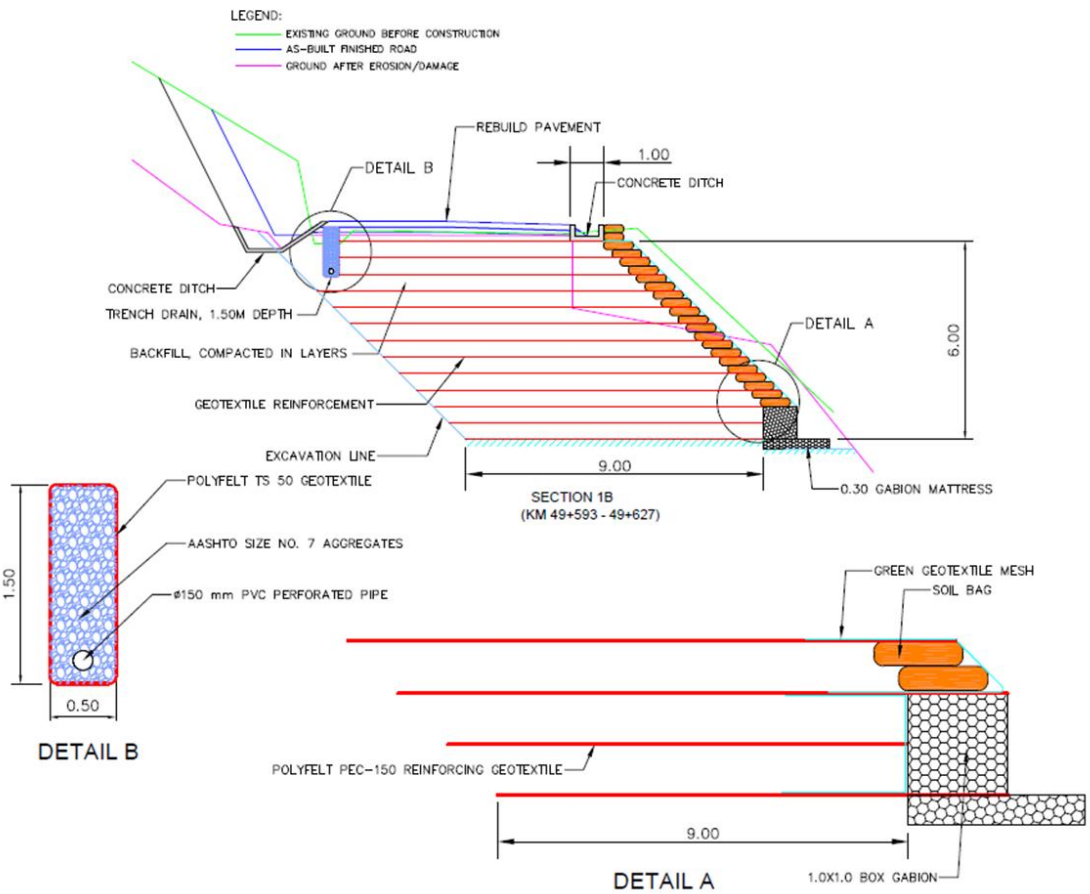

Fig. 25 Mitigation scheme combining trench drain and reinforcement with high strength geotextiles

6. CONCLUSIONS

The following conclusions can be drawn from the aforementioned presentations of rain-trigged landslides mitigations using construction methods with geosynthetics:

1) During the recent past, landslides caused by heavy rains have bought loss of lives and significant damages of properties as well as infrastructures in Thailand and around the world. While extreme weather conditions have been identified as triggering causal factor of landslides, man-made disturbance of natural slopes such as deforestations and highway construction, which change the surface and underground water flow characteristics, are also contributing factors.

2) From the full scale test, the lateral deformations of Reinforced Steep Slope (RSS) with polymer geogrid reinforcement facing were much more compared to the Mechanically Stabilized Earth Wall (MSEW) with metallic reinforcements due to the lower stiffness of the former than the latter. The comparisons of stiffness of the reinforcing materials in descending order were metallic strip (MS), steel wire grid (SWG), polypropylene (PP), high density polyethylene (HDPE) and polyester (PET). The lateral deformations for both RSS and MSEW facing with different types of geogrid reinforcements obtained from PLAXIS 3D simulations were in good agreement with the corresponding observed data.

3) Geosynthetics have been successfully applied and incorporated in construction methods in National Road (NR 1B) in Laos for mitigations of rain-triggered landslides. However, there are lateral deformations which need to be addressed.

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