Article

Natural Para Rubber in Road Embankment Stabilization

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Abstract: This is the first study on “ribbed smoked sheets (RSS)” as a geogrid reinforcement in geotechnical engineering. An RSS is a kind of natural para rubber. RSS (grade 3) was designed as a biaxial geogrid with an aperture size of 20 mm × 20 mm and a spacing of 20 mm. The RSS was found to be a significant functional layer when applied to the subbase lateritic soil layer. The lateritic soil with an RSS reinforcing layer was greatly improved regarding the California bearing ratio (CBR). Numerical simulation using two-dimensional finite element software was used to determine the optimal number and positions of the RSS reinforcing layers in road embankment stabilization. The simulation data in terms of horizontal displacement of unreinforced road embankments was validated by the collected data from the actual construction site. The RSS reinforced layer was varied from one to three layers under 61 analysis conditions. The highest safety factor was obtained with two layers of RSS at 0.1H below the top of the road embankment and 0.4H below the first RSS layer, suggesting a suitable installation of the RSS reinforcing layer. The RSS is thus strongly recommended as a reinforcing material in low CBR lateritic soil for the road embankment.

Keywords: geosynthetics; geogrid; RSS; para rubber sheet; ribbed smoked sheet; road reinforcement

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1. Introduction

A road embankment, which is the soil layer, can withstand strong compression but the tension property is very weak. The reinforcement material, which is stiffer than the road material, has become one alternative to apply for increasing stability, bearing capacity, and strength of soil, especially in road construction [1,2]. Geogrid, which is made from polymer materials, such as polypropylene, polyester, high-density polyethylene, or other polymers with high elastic properties, have been widely applied and placed in road embankments to improve the performance of weak subgrades and the base course for both paved and unpaved roads [3]. The biaxial geogrid is generally used for road reinforcement to distribute a load in two directions, which are the machine direction (MD) and cross direction (CD). The geogrid produces interlocking between aggregates and the geogrid [4], which creates an aggregate–geogrid composite [5] to form a confined zone with three functions (lateral restraint, bearing capacity increment, and membrane support). With the presence of the interlocking mechanisms, the geogrid can improve the lateral restraint at the interface and prevent both lateral and vertical movements. The higher shear strength surface is developed by using geogrid reinforcement to increase the bearing capacity and support the wheel load as a membrane support [4,6]. The biaxial geogrid can be applied to reduce the base course thickness by 30% [7]. The thickness of a road embankment for both paved and unpaved surfaces can be reduced by 20% to 70% depending on the strength of the subgrade and the type of geogrid [8]. Therefore, geogrid reinforcement can increase the bearing capacity, reduce rutting, and decrease the settlement [9].

The performance of geogrid reinforcement depends on the reinforcing layer and the reinforcing position. The position of the geogrid reinforcement depends on various parameters, which play an important role, such as the distance of the topmost layer from the loading bed (u), the distance between geogrid layers (h), the loading area width (B), the width of the geogrid layers (b), the number of geogrid layers (N), the loading distance from the edge of slope (D), and the slope angle (β). The bearing capacity increases with an increase in the reinforcing layers within the road embankment [10]. In some cases, geogrid reinforcement with a single layer is sufficient if the depth of the reinforcing position is appropriate [11]. El Sawwaf [12] found that the optimal amount of the reinforcing position for u/b and h/b were 0.6 and 0.5, respectively. Alamshahi and Hataf [13] proposed that having two layers of a geogrid-reinforced slope is the optimal number of geogrid reinforcements based on the bearing capacity. Esmaeili et al. [14] investigated the optimal position for geogrid reinforcement, which led to the best performance of the geogrid reinforcement by considering the maximum bearing capacity of the railway embankment; the optimized results were the normalized distances $u/B = 0.3$ and $h/B = 0.4$ with three layers of the geogrid reinforcement, based on B , u , and h being 4.8 m, 1.44 m, and 1.92 m, respectively. Moreover, four layers of the geogrid reinforcement did not increase the bearing capacity of the railway embankment.

In recent years, biodegradable- or natural-reinforced materials have been investigated to increase their applications in geotechnical engineering [15,16]. Currently, para rubber prices are experiencing a problem in Thailand. Moreover, the natural para rubber is interesting not only because of its superior properties but also to overcome the problem of supply and price in Thailand. Natural rubber is a biopolymer consisting of isoprene units $(C_5H_8)_n$ linked together in a 1,4 cis-configuration. Not only poly cis-isoprene molecules but also other cellular components are included in the natural rubber, while synthetic rubber is almost completely composed of polyisoprene with not more than 90% of cis-bond. Because of its molecular structure with a high cis-bond content of over 99.5% and high molecular weight, the natural rubber has superior resilience, elasticity, abrasion resistance, efficient heat dispersion, and impact resistance to synthetic rubber [17–19]. Para rubber can be used to produce latex, block rubber, RSS, etc. Para rubber was researched by many researchers in the forms of para rubber floor tiles [20], admixture in concrete blocks [21], para soil-cement road [22], etc. Concentrated latex was suggested to mix with soil cement because it can improve the properties of soil-cement roads, such as an increase in compression, strength, and bending strength, while reducing absorption and dust [23]. However, research on using RSS reinforcement for its geogrid function has yet to be published.

Therefore, preliminary research was conducted on the application of RSS as a reinforced material to improve the strength of lateritic soil. Moreover, this research aimed to investigate the optimal layer and position of RSS reinforcement on the road embankment, which was located on soft ground areas based on safety analysis conducted using numerical simulation.

2. Properties of RSS

RSS is made from coagulated latex sheets, which are smoked in an oven at a suitable temperature. According to the green book standard by the Rubber Authority of Thailand [24], RSS is classified into six quality grades, which are RSS grades 1X, 1, 2, 3, 4, and 5. The best quality is RSS grade 1X, with strict quality control regarding the purity, elasticity, colors, air bubbles, defections, shape, and moisture. On the other hand, the worst quality is RSS grade 5 [24].

In this research, RSS grade 3 was designed as a biaxial geogrid in the shape of a square with an aperture size of 20 mm $\times$ 20 mm and a spacing of 20 mm, as shown in Figure 1. The apertures can interlock with surrounding aggregates, forming a confined zone and allowing for the draining of water [6]. The aperture size was designed based on the research of Koerner [6] and Han et al. [25], which suggested that the size of a geogrid aperture should be between 10 mm and 100 mm, with an open area percentage between 40% and

90%. Due to the efficiency of interlocking, the ratio of aperture size (A) to particle diameter size (D) of a biaxial geogrid, namely, A/D , should be between 1.30–1.71 [25].

Figure 1. Apertures size of RSS for road reinforcement.

According to ASTM D5199-12 [26], the thickness of the RSS was approximately 2.90 mm to 3.90 mm for a single sheet and approximately 7.20 mm to 7.60 mm for a double sheet. The ultimate tensile strength of RSS was measured using the wide-width strip method [27]. A universal testing machine (Instron 5567) at a speed of 10 mm/min (10% of strain rate) with a gauge length of 100 mm was used for the testing. The load was applied to the RSS until failure. As a result, the ultimate tensile strength of the RSS was 0.64 kN/m and 1.313 kN/m for a single sheet and a double sheet, respectively. The ultimate tensile strength of the RSS was within the typical range of geogrid [28].

3. Preliminary Study of RSS as Reinforcing Materials

The performance of RSS with a geogrid function was evaluated in the laboratory by measuring the strength of the lateritic soil based on the California Bearing Ratio (CBR) test. According to the DH-S 205/2532 standard [29] from the Thailand Department of Highways, the classification of this grade of lateritic soil is appropriate for application as a subbase material based on the grain size distribution shown in Table 1. Lateritic soil is classified into five grades: A, B, C, D, and E. The first order grade is A, with the best lateritic soil properties with the highest maximum dry density and CBR values but the lowest optimal water content value. Meanwhile, the last order grade is E with lateritic soil with the lowest maximum dry density and CBR but the highest optimal water content [30].

To determine the performance of RSS reinforcement on the CBR of lateritic soil, the lateritic soil grade E was used in this research. Moreover, the lateritic soil grade E was classified into two gradations to understand the RSS performance based on differences in subsoil strength. For lateritic soil grade E with a high CBR (CBR = 64.53%), which was called E1, the percent passing through sieve nos. 10, 40, and 200 of the lateritic soil was 70.00 wt%, 31.00 wt%, and 6.00 wt%, respectively. For lateritic soil grade E with low CBR (CBR = 40.56%), which was called E2, the percent passing through sieve nos. 10, 40, and 200 of the lateritic soil was 75.67 wt%, 37.10 wt%, and 13.51 wt%, respectively. Gradation of lateritic soil was performed via sieve analysis in order to determine the grain size distribution, as shown in Table 1 and Figure 2.

Table 1. Grain size distributions for lateritic soil.

Sieve Size	Percent Passing by Weight (%)						
	Standard of DH-S 205/2532					In This Research	
	Grade A	Grade B	Grade C	Grade D	Grade E	Grade E1	Grade E2
2" (50.00 mm)	100	100	-	-	-	-	-
1" (25.00 mm)	-	-	100	100	100	100	100
3/8" (9.50 mm)	30–65	40–75	50–85	60–100	-	-	-
No. 10 (2.00 mm)	15–40	20–45	25–50	40–70	40–100	70	75.67
No. 40 (0.425 mm)	8–20	15–30	15–30	25–45	20–50	31	37.10
No. 200 (0.075 mm)	2–8	5–20	5–15	5–20	6–20	6	13.51

Figure 2. Grain sizes distribution of E1 and E2.

The liquid limit (LL), plastic limit (PL), and plasticity index (PI) of the lateritic soil were characterized using Atterberg's limits test based on the AASHTO T 89 [31] and AASHTO T 90 [32] standards. The LL and PL of E1 were 48.00% and 21.54%, respectively. The PI, which was LL-PL, of E1 was 26.46%. Moreover, The LL and PL of E2 were 48.00% and 23.23%, respectively. The PI of E2 was 24.77%. The average optimum moisture content (OMC) of lateritic soil was found to be 14% at a maximum dry density of 1.86 g/cm³.

CBR Testing

CBR testing was carried out and compared under four testing conditions, which were lateritic soil E1 non-reinforcement (E1), lateritic soil E2 non-reinforcement (E2), lateritic soil E1 with RSS reinforcement (E1-RSS), and lateritic soil E2 with RSS reinforcement (E2-RSS), respectively.

The standard for the CBR testing method, namely, ASTM D1883 [33], was tested based on the condition of OMC at maximum dry density, which was obtained from conducting the modified compaction test [34]. The maximum dry density of E1 and E2 was 1.86 g/cm³ at the OMC of 14.00%. A CBR under non-reinforcement conditions (E1 and E2) with lateritic soils E1 and E2 were mixed with tap water until reaching homogeneity. The soil samples were transferred into CBR molds (cylindrical molds with a 152.4 mm inner diameter and a height of 177.8 mm) in sequential order with five equal-sized soil sample amounts reaching up to the top edge of the mold. Each layer of soil sample was compacted with 56 blows using a 10.00 lbf (44.48 N) rammer, which was dropped from a 457.2 mm height. After compaction, the perforated base plate and spacer disk were taken out. The molds containing compacted soil samples were weighed and recorded. A disk of coarse filter paper was placed on the top of the soil sample prior to overturning the CBR mold. In penetration testing, a steel penetration piston with a 50 mm diameter proving ring

was inserted at the center point for penetration. Load measurements corresponding to deformation at every 0.64 mm (0.025 in) were taken to calculate the bearing capacity of the lateritic soils E1 and E2.

A CBR test under reinforcement conditions (E1-RSS and E2-RSS) followed the same preparation procedure with non-reinforcement conditions (E1 and E2). On the other hand, the RSS with an aperture size of 20 mm $\times$ 20 mm was cut as a circular shape and placed between layers 2 and 3 of the soil samples in the CBR mold, as shown in Figure 3.

Figure 3. The position of RSS in the CBR mold.

To determine the effect of RSS reinforcement on the CBR of lateritic soil, E1 (CBR = 64.53%) and E2 (CBR = 40.56%) were representative of lateritic soil in the cases of high CBR and low CBR, respectively. By considering the effect of RSS-reinforced lateritic soil with a high CBR value, E1 and E1-RSS were compared as shown in Table 2. The CBR value in the case of E1-RSS (CBR = 64.31%) was lower than the case of E1 (CBR = 64.53%). This result showed that in cases with RSS reinforcement, the CBR of the lateritic soil was decreased by 0.34%. Meanwhile, E2 and E2-RSS were compared to determine the effect of RSS-reinforced lateritic soil on a low CBR value. The CBR value in the case of E2-RSS (CBR = 49.95%) was higher than the E2 case (CBR = 40.56%), with a percentage difference of 23.15% as presented in Table 2. Therefore, the RSS was strongly recommended to improve the strength of lateritic soil, which had low CBR values. In the case of RSS reinforcement for soil with high CBR values, it is not recommended as a reinforcing material because it acts as a weak layer in hard soil according to the results of geogrid reinforcement, which are more efficient for low-CBR soil [6,35–37].

Table 2. The comparison of CBR values in RSS reinforcement.

Soil Types	Testing Conditions		CBR (%)	Percentage Difference (%)
High CBR	Non-reinforced	(E1)	64.53	−0.34(Decreased)
	Reinforced 1 layer	(E1-RSS)	64.31	
Low CBR	Non-reinforced	(E2)	40.56	+23.15(Increased)
	Reinforced 1 layer	(E2-RSS)	49.95	

4. Numerical Simulation

The finite element method (FEM) was chosen to confirm the number of effective layers and the position of the RSS-reinforced road embankment. PLAXIS (American Bentley Systems, Inc., Exton, PA, USA) ennsylvania is the most reliable software for FEM modeling and solving problems in geotechnical work using two-dimensional (2D) FEM under plane

strain conditions [38,39]. Moreover, several previous works revealed that the simulation results from PLAXIS software are aligned with the actual situation in complex ground conditions [40–42]. PLAXIS software is therefore suitable for simulating the geometry of the road embankment. The typical geometry of canal-side roads and subsoils were modeled as the condition of an unreinforced road embankment. A nominal surcharge of 10 kN/m was assigned for traffic-load modeling. The condition of a very fine mesh was generated in the FEM model. The construction stage feature allowed for incremental backfill placement, which was divided into three layers. The RSS reinforcement was placed on the top of a compacted backfill soil layer-by-layer until reaching the full height of the road embankment at 2 m. The water level was assigned at 1.5 m below the ground surface. The in-situ stresses in the subsoil layer were generated using the K_0 procedure. The compacted backfill soil as an additional layer was assigned as a material parameter according to the stress state induced after an incremental backfill layer. During the construction stage, undrained analysis was applied to simulate the construction of the road embankment. After completion of the full height of the road embankment, drainage analysis was applied to simulate the consolidation process for the road embankment. Moreover, the subsoil model was created and the results from the FEM software were validated based on horizontal displacement profiles, which were measured in the real construction site at the Ayutthaya rural road no. 5034 (AY.5034).

4.1. Validation Site

AY.5034 is a canal-side road, located on soft Bangkok clay in the Don Thong subdistrict, Sena district, Phra Nakhon Si Ayutthaya, Thailand. AY.5034 has asphaltic concrete with a two-lane road (section of 6 m wide and a 1 m wide shoulder on each side), consisting of one lane in one direction and one lane in the other. The side slope of the canal-side road is 1V:1.5H. The depth of the canal is about 4 m below the ground surface and the height of the road embankment is about 2 m high from the ground surface. The water level in the canal of the existing road was at a 1.5 m depth below the ground surface. The general soil profile and soil properties were investigated at KM. 6 + 200, as shown in Figure 4. The topsoil was about 1.5 m in depth. The uppermost 15 m in depth in the subsoil could be divided into six layers. The first layer was the 1.5 m thick medium clay, which was a dark grey color. The second was the very soft to soft clay layer down to 7.5 m in depth. The third layer was a stiff clay layer with a medium-light grey color at 7.5 to 9.0 m in depth. The fourth layer was stiff sandy clay ranging from 9.0 to 10.5 m in depth. The very dense silty yellowish-gray sand down to 12.0 m in depth formed the fifth layer and the medium dense to dense yellowish-gray silty sand ranging from 12.0 to 15.0 m in depth formed the sixth layer. Underlying the hard clay was sand and hard sandy clay, which extended to 19 m in depth.

Figure 4. Soil profile and soil properties at AY.5034.

4.2. Model Geometry and Materials Properties

4.2.1. Geometry of Road Embankment

The geometry of the road embankment was presented using a cross-section of AY.5034, which was constructed along the Khong-Khun Si (irrigation canal). This kind of geometry was suitable for simulating using a 2D under-plane strain condition with a 15-node element. To avoid any boundary effects, the FEM model was created with a 330 m width in the x -axis (from $x_{\min}$ of -175 m to $x_{\max}$ of 155 m) and 64 m in depth in the y -axis (from $y_{\min}$ of -60 m to $y_{\max}$ of 4 m), which is enough boundary size [43].

The road embankment and the canal were modeled with a 2 m height and 4 m depth from the ground surface. The water level in the canal was assigned at 1.5 m below the ground surface. The subsoil layer model was divided into 11 layers based on the soil profile of AY.5034, as shown in Figure 5. The roller and fixed boundaries were set to the side and the bottom of the model boundaries, respectively.

Figure 5. FEM model.

4.2.2. Subsoil and Road Embankment Properties in FEM Model

The model parameter was assigned based on the soil profile of AY.5034 (Figure 4). The soft soil model (SSM) under undrained conditions was assigned to the uppermost 7.5 m in depth with an overconsolidation ratio (OCR) = 1.2, which included the topsoil, medium clay, and soft clay layers. The strength parameters of the topsoil were cohesion $c' = 14$ kPa and friction angle $\phi' = 25^\circ$. Additional values for analysis were as follows: modified compression index $\lambda^* = 0.17$ and modified swelling index $\kappa^* = 0.020$. The medium clay (1.5 m to 3.0 m in depth) with $c' = 15$ kPa, $\phi' = 25^\circ$, $\lambda^* = 0.12$, and $\kappa^* = 0.020$ [44].

The soft soil layer (3.0 m to 7.5 m in depth) could be divided into three sublayers, namely, very soft to soft clay 1 (3.0 m to 4.5 m in depth), very soft to soft clay 2 (4.5 m to 6.0 m in depth) and very soft to soft clay 3 (6.0 m to 7.5 m in depth). The strength parameters were determined based on the testing data at AY.5304 by using strength parameters c' between 10 and 14 kPa and $\phi' = 25^\circ$. The additional values for analysis were as follows: $\lambda^* = 0.10$ for very soft to soft clay 1 and 0.03 for very soft to soft clay 2 and 3, and $\kappa^* = 0.009$ for the soft soil layer [44].

The elastic perfectly plastic Mohr–Coulomb model (MCM) was assigned for the model of underlying stiff clay to hard sandy clay, which was located from 7.5 m to 19 m in depth. The stiff clay was located between 7.5 m and 9.0 m in depth. The strength parameters with the undrained condition were obtained from the validation site with $c' = 95$ kPa, the Poisson's ratio $\nu' = 0.25$, and elasticity $E' = 19,000$ kPa. The strength parameters with the undrained condition of stiff sandy clay layer (9.0 m to 10.5 m depths) were $c' = 105$ kPa, $\phi' = 10^\circ$, $\nu' = 0.25$, and $E' = 21,000$ kPa. The very dense silty sand (10.5 m to 12.0 m depths) and medium dense to dense silty sand (12.0 m to 15.0 m in depth) with the drained

condition were assigned to the FEM model with constant values for $c' = 5$ kPa, $v' = 0.30$, and $E' = 53,000$ kPa, while the value of $\phi' = 40^\circ$ and 35° for very dense silty sand and medium dense to dense silty sand, respectively.

The FEM input parameters of hard clay with sand (15.0 m to 16.5 m in depth) in the analysis were as follows: $c' = 150$ kPa, $\phi' = 10^\circ$, $E' = 30,000$ kPa, and $v' = 0.25$. Hard sandy clay was the final layer that was located from 15 m to 60 m in depth. The model parameters used for the FEM analysis were as follows: $E' = 54,000$ kPa, $v' = 0.25$, $c' = 270$ kPa, and $\phi' = 10^\circ$. The modulus of elasticity and strength parameters for the subsoil layers, which were generated as a foundation, are presented in Table 3.

Moreover, the material properties of the road embankment for the FEM analysis were established from a previous study by Wulandari and Tjandra [45], which analyzed a geotextile reinforced road embankment using PLAXIS 2D. The MCM with drained behavior was used for the model of a road embankment. The strength parameters were $c' = 1$ kPa, $\phi' = 33^\circ$, and the dilatancy angle $\psi = 3^\circ$. The additional material parameters for FEM analyzes were $E' = 50,000$ kPa and $\nu' = 0.3$. The interface coefficient $R_{in} = 0.8$ [46]. The material properties of the road embankment model for the FEM simulations are tabulated in Table 3.

4.2.3. RSS Properties in FEM Model

The RSS was modeled as a geogrid element in the FEM software. An axial stiffness (EA), which is the elastic modulus of the reinforcement material (E) and the cross-sectional area per unit width in the perpendicular plane (A), is the required input value for the geogrid element. The EA was calculated from the ultimate tensile strength of RSS, which was obtained using the wide-width strip method [27]. The double-sheet of RSS, which was 7.6 mm in thickness, was applied as the thickness of the reinforcement material in the FEM model. The ultimate tensile strength (T_{ult}) was 1.313 kN/m, which is typical of the peak tensile strength range [28]. Therefore, EA was 3.46 kN/m at a strain of 25%. The properties of the RSS used in this analysis are tabulated in Table 4.

Table 3. The material properties used in the FEM analysis of the road embankment with RSS reinforcement.

Materials	Depth (m)	Model	Behaviors	γ_{sat} (kN/m ³)	γ_{un} (kN/m ³)	E'_{ref} (kPa)	v'	ν^*	κ^*	c' (kPa)	ϕ' (°)	OCR
Subsoil												
Top soil	0–1.5	SSM	Undrained	15.0	13.0			0.17	0.020	14	25	1.2
Medium clay	1.5–3.0	SSM	Undrained	16.5	14.5			0.12	0.020	15	25	1.2
Very soft to soft clay 1	3.0–4.5	SSM	Undrained	15.0	13.0			0.10	0.009	14	25	1.2
Very soft to soft clay 2	4.5–6.0	SSM	Undrained	15.0	13.0			0.03	0.009	10	25	1.2
Very soft to soft clay 3	6.0–7.5	SSM	Undrained	15.0	13.0			0.03	0.009	10	25	1.2
Stiff clay	7.5–9.0	MCM	Undrained	21.0	19.0	19,000	0.25			95		
Stiff sandy clay	9.0–10.5	MCM	Undrained	21.0	19.0	21,000	0.25			105	10	
Very dense silty sand	10.5–12.0	MCM	Drained	22.0	20.0	53,000	0.30			5	40	
Medium dense to dense silty sand	12.0–15.0	MCM	Drained	22.0	20.0	53,000	0.30			5	35	
Hard clay with sand	15.0–16.5	MCM	Undrained	21.0	19.0	30,000	0.25			150	10	
Hard sandy clay	16.5–19.0	MCM	Undrained	22.0	20.0	54,000	0.25			270	10	
Road Embankment [45]												
Materials	Model	Behaviors	γ_{sat} (kN/m ³)	γ_{un} (kN/m ³)	E'_{ref} (kPa)	v'	ψ (°)	R_{in}				
Fill material	MCM	Drained	18.0	18.00	50,000	0.3	3.00	0.8				

Table 4. The properties of the RSS in the double sheet condition.

Material	Mesh Type	Apertures Size (mm)	Thickness (mm)	Ultimate Tensile Strength, T_{ult} (kN/m)	Axial Stiffness, EA (kN/m)
RSS—geogrid	Rectangular apertures	20 × 20	7.591	1.313	3.460

5. Variable Assignment in FEM Model

To study the optimal layer and position of the RSS reinforcement on a road embankment considering the factor of safety, the parameters of the reinforced road embankment were applied through parametric analysis. The FEM model considered the following parameters: distance from the topmost reinforcing layer to the road embankment surface (u), spacing between the reinforcing layers (h), height of the road embankment above the subsoil level (H), width of the road embankment (B), and the number of reinforcing layers (N), as shown in Figure 6.

Figure 6. The parameters of road embankment reinforced with RSS geogrid layers.

5.1. The Concept of Reinforcement Conditions

The concept of u/H and $h(N - 1)/H$ were utilized to analyze optimal reinforcing layers and reinforcing positions for the road embankment reinforced using the RSS. This concept depended on the following parameters: u , h , H , and N of a reinforced road embankment. The general equation has been approved, as shown in Equation (1):

$$u + (N - 1)(h) \leq H \quad (1)$$

For one layer of RSS reinforcement ($N = 1$):

$$u + (0)(h) \leq H \quad (2)$$

$$u_{\min} = 0H, u_{\max} = 1H:$$

$$\therefore u/H \leq 1 \quad (3)$$

For two layers of RSS reinforcement ($N = 2$):

$$u + (1)(h) \leq H \quad (4)$$

$$u_{\min} = 0H, u_{\max} = 0.5H; (1)h_{\min} = 0H, (1)h_{\max} = 0.5H.$$

For three layers of RSS reinforcement ($N = 3$):

$$u + (2)(h) \leq H \quad (5)$$

$$u_{\min} = 0H, u_{\max} = 0.5H; (2)h_{\min} = 0H, (2)h_{\max} = 0.5H:$$

$$\therefore N = 2, \text{ and } N = 3; u/H \leq 0.5 \quad (6)$$

$$h(N - 1)/H \leq 0.5 \quad (7)$$

Therefore, from the concept of u/H and $h(N - 1)/H$, it was concluded that $u/H \leq 1$ (Equation (3)) should be applied for $N = 1$, while $u/H \leq 0.5$ (Equation (6)) and $h(N - 1)/H \leq 0.5$ (Equation (7)) should be applied for $N = 2$ and 3.

The limitations of the u/H and $h(N - 1)/H$ concepts are as follows:

- (1) A surcharge load acts on the full width of a road embankment (for this study, a surcharge load = 10 kN/m);
- (2) H is constant (for this study, H = 2.00 m height);
- (3) u, h, and N are different in different positions;
- (4) The effect of a side slope is ignored.

5.2. Analysis Conditions

The 2D FEM analysis was carried out to evaluate and confirm the efficiency of the RSS-reinforced road embankment and determine the reinforcing layers and reinforcing positions, comparing between the unreinforced (N = 0) and reinforced (N = 1, 2, 3) conditions with 61 analysis conditions, as shown in Table 5.

Table 5. The 61 analysis conditions.

No. of RSS Layers, N	Conditions	Reinforcing Positions						
		Spacing		From the Road Surface		From the Ground Surface (m)		
		h(N – 1)	h(m)	u	u (m)	N1	N2	N3
N = 0	N0	-	-	-	-	-	-	-
N = 1	N1-u ₁	-	-	0.1H	0.20	1.80	-	-
	N1-u ₂	-	-	0.2H	0.40	1.60	-	-
	N1-u ₃	-	-	0.3H	0.60	1.40	-	-
	N1-u ₄	-	-	0.4H	0.80	1.20	-	-
	N1-u ₅	-	-	0.5H	1.00	1.00	-	-
	N1-u ₆	-	-	0.6H	1.20	0.80	-	-
	N1-u ₇	-	-	0.7H	1.40	0.60	-	-
	N1-u ₈	-	-	0.8H	1.60	0.40	-	-
	N1-u ₉	-	-	0.9H	1.80	0.20	-	-
	N1-u ₁₀	-	-	1.0H	2.00	0.00	-	-
N = 2	N2-h ₁ u ₁	0.1H	0.20	0.1H	0.20	1.80	1.60	-
	N2-h ₁ u ₂			0.2H	0.40	1.60	1.40	-
	N2-h ₁ u ₃			0.3H	0.60	1.40	1.20	-
	N2-h ₁ u ₄			0.4H	0.80	1.20	1.00	-
	N2-h ₁ u ₅			0.5H	1.00	1.00	0.80	-
	N2-h ₂ u ₁	0.2H	0.40	0.1H	0.20	1.80	1.40	-
	N2-h ₂ u ₂			0.2H	0.40	1.60	1.20	-
	N2-h ₂ u ₃			0.3H	0.60	1.40	1.00	-
	N2-h ₂ u ₄			0.4H	0.80	1.20	0.80	-
	N2-h ₂ u ₅			0.5H	1.00	1.00	0.60	-
	N2-h ₃ u ₁	0.3H	0.60	0.1H	0.20	1.80	1.20	-
	N2-h ₃ u ₂			0.2H	0.40	1.60	1.00	-
	N2-h ₃ u ₃			0.3H	0.60	1.40	0.80	-
	N2-h ₃ u ₄			0.4H	0.80	1.20	0.60	-
	N2-h ₃ u ₅			0.5H	1.00	1.00	0.40	-
	N2-h ₄ u ₁	0.4H	0.80	0.1H	0.20	1.80	1.00	-
	N2-h ₄ u ₂			0.2H	0.40	1.60	0.80	-
	N2-h ₄ u ₃			0.3H	0.60	1.40	0.60	-
	N2-h ₄ u ₄			0.4H	0.80	1.20	0.40	-
	N2-h ₄ u ₅			0.5H	1.00	1.00	0.20	-
	N2-h ₅ u ₁	0.5H	1.00	0.1H	0.20	1.80	0.80	-
	N2-h ₅ u ₂			0.2H	0.40	1.60	0.60	-
	N2-h ₅ u ₃			0.3H	0.60	1.40	0.40	-
	N2-h ₅ u ₄			0.4H	0.80	1.20	0.20	-
	N2-h ₅ u ₅			0.5H	1.00	1.00	0.00	-

Table 5. Cont.

No. of RSS Layers, N	Conditions	Reinforcing Positions						
		Spacing		From the Road Surface		From the Ground Surface (m)		
		h(N – 1)	h(m)	u	u (m)	N1	N2	N3
N = 3	N3-h ₁ u ₁	0.1H	0.10	0.1H	0.20	1.80	1.70	1.60
	N3-h ₁ u ₂			0.2H	0.40	1.60	1.50	1.40
	N3-h ₁ u ₃			0.3H	0.60	1.40	1.30	1.20
	N3-h ₁ u ₄			0.4H	0.80	1.20	1.10	1.00
	N3-h ₁ u ₅			0.5H	1.00	1.00	0.90	0.80
	N3-h ₂ u ₁	0.2H	0.20	0.1H	0.20	1.80	1.60	1.40
	N3-h ₂ u ₂			0.2H	0.40	1.60	1.40	1.20
	N3-h ₂ u ₃			0.3H	0.60	1.40	1.20	1.00
	N3-h ₂ u ₄			0.4H	0.80	1.20	1.00	0.80
	N3-h ₂ u ₅			0.5H	1.00	1.00	0.80	0.60
	N3-h ₃ u ₁	0.3H	0.30	0.1H	0.20	1.80	1.50	1.20
	N3-h ₃ u ₂			0.2H	0.40	1.60	1.30	1.00
	N3-h ₃ u ₃			0.3H	0.60	1.40	1.10	0.80
	N3-h ₃ u ₄			0.4H	0.80	1.20	0.90	0.60
	N3-h ₃ u ₅			0.5H	1.00	1.00	0.70	0.40
	N3-h ₄ u ₁	0.4H	0.40	0.1H	0.20	1.80	1.40	1.00
	N3-h ₄ u ₂			0.2H	0.40	1.60	1.20	0.80
	N3-h ₄ u ₃			0.3H	0.60	1.40	1.00	0.60
	N3-h ₄ u ₄			0.4H	0.80	1.20	0.80	0.40
	N3-h ₄ u ₅			0.5H	1.00	1.00	0.60	0.20
	N3-h ₅ u ₁	0.5H	0.50	0.1H	0.20	1.80	1.30	0.80
	N3-h ₅ u ₂			0.2H	0.40	1.60	1.10	0.60
	N3-h ₅ u ₃			0.3H	0.60	1.40	0.90	0.40
	N3-h ₅ u ₄			0.4H	0.80	1.20	0.70	0.20
	N3-h ₅ u ₅			0.5H	1.00	1.00	0.50	0.00

5.3. Factor of Safety Analysis

The factor of safety (FS) was used to analyze and confirm the optimal layer and position of RSS reinforcement on the road embankment. The results of the FS, which was analyzed using the PLAXIS program, were calculated based on the phi-c reduction method [47]. The phi-c reduction method was calculated by reducing the soil strength parameters ($\tan\phi$ and c) until the failure of the road embankment or subsoil layer occurs. The total multiplier (ΣM_{sf}) was used to define the value of the soil-strength parameters at a given stage in the FEM analysis, as in Equation (8):

$$\Sigma M_{sf} = \frac{\tan \phi_{\text{input}}}{\tan \phi_{\text{reduced}}} = \frac{C_{\text{input}}}{C_{\text{reduced}}} \quad (8)$$

where the soil strength parameters with the subscript “input” refer to the properties entered in the material sets, while the soil strength parameters with the subscript “reduced” refer to the reduced values used in the FEM analysis. ΣM_{sf} was set to 1.0 at the start of a calculation step to set all material strengths to their unreduced values. The soil strength parameters were successively reduced automatically until failure occurred. The FS is given by Equation (9):

$$FS = \frac{\text{Available strength}}{\text{Strength at failure}} = \text{Value of } \Sigma M_{sf} \text{ at failure} \quad (9)$$

6. Analysis of Results

6.1. Model Validation

The subsoil model was validated by using horizontal displacement profiles (u_x), which were measured at a real construction site at AY.5034 (KM. 5 + 400). The inclinometer was installed on 6 June 2018 to measure the u_x at a 16 m depth below the ground's surface. For model validation, the u_x of the subsoil layer at 462 days (11 September 2020) after installation from the AY.5034 was measured and compared with the u_x from the FEM model under the following analysis conditions: (1) the road embankment model with a nominal surcharge of 10 kN/m were assigned at 462 days and (2) the u_x from the FEM model was considered at the coordinates of $(-5,0)$ to $(-5,-16)$ at the same position as the inclinometer at AY.5034.

The validation graph shows the relationship between the u_x in the x -axis and the subsoil depth in the y -axis. The maximum u_x occurred at the ground's surface, which was 34.39 mm with a rate of 2.99 mm/month. For the uppermost 5 m depth below the ground's surface, u_x clearly occurred and it continuously reduced with an increase in the depth of subsoil. On the other hand, the u_x slightly occurred underlying a depth of 5 m below the ground surface, which went down from 5 m to 11 m depths. At depths more than 11 m below the ground's surface, u_x did not occur at AY.5034. Moreover, the results of u_x from the FEM analysis followed a similar trend while measuring u_x at AY.5034. This result could be used to confirm that the input parameter on the FEM model was reliable, as shown in Figure 7.

Figure 7. The validation data.

6.2. Optimal Layer and Position of RSS Reinforced Road Embankment

Comparing the FS of the unreinforced road embankment ($N = 0$) and reinforced road embankment with one layer of RSS ($N = 1$), the FS of $N = 0$ was 1.255. The FS of $N = 1$ continuously reduced with an increase in u/H . The FS of the reinforced road embankment was higher than that of the unreinforced road embankment. This means that the RSS was appropriate to install in the structure of the road embankment as the reinforcement material. In addition, the FS of $N = 1$ at $u/H = 0.1$, which was called $N1-u_1$, had a maximum FS at 1.415, as shown in Figure 8.

Figure 8. The FS results in the conditions of $N = 0$ and $N = 1$.

Figure 9 presents the relationship between FS and $h(N - 1)/H$ of the reinforced road embankment with two layers of RSS ($N = 2$) by varying u/H from 0.1 to 0.5. The FS increased with an increase of u/H , which had the highest FS at $u/H = 0.1$ and the lowest FS at $u/H = 0.1$. Reducing $h(N - 1)/H$ did not have much of an effect on the FS in the case of $u/H = 0.2$ to 0.5. On the other hand, the FS of $u/H = 0.1$ showed a significant increase in the decrease of $h(N - 1)/H$. This result followed a similar trend to the results of the reinforced road embankment with three layers of RSS ($N = 3$), as shown in Figure 10.

Figure 9. The FS results in the conditions with $N = 2$.

According to the results of FS in the case of $N = 1$ to $N = 3$, it confirmed that the optimal position of RSS reinforced road embankment in the top layer was $u/H = 0.1$. The relationship between FS and $h(N - 1)/H$ in the condition of $N = 2$ and $N = 3$ were plotted and compared to analyze the optimization of reinforcing layers, as shown in Figure 11. The FS in the condition of $N = 2$ was higher than the FS in the condition of $N = 3$ for all analyzed $h(N - 1)/H$ conditions.

Therefore, two layers of RSS-reinforced road embankment were optimal. This result is consistent with the suggestion of Alamshahi and Hataf [13], which mentioned that two layers of reinforcement material were suitable to increase the bearing capacity of the embankment. Moreover, Esmaili et al. [14] mentioned that four layers of reinforcement material have no effect on increasing the bearing capacity. The FS of $h(N - 1)/H = 0.4$ was 1.475, which was the maximum FS. The results of the analysis were similar to the results from the geogrid stabilized high railway embankment from Esmaili et al. [14]. Thus, the

$N2-h_4u_1$ condition, which included two RSS layers with the first layer of RSS at $0.1H$ below the road surface and the second layer of RSS at $0.4H$ below the position of the RSS in the first layer, provided the optimal number of layers and their positions for the reinforced road embankment when using RSS.

Figure 10. The FS results in the conditions with $N = 3$.

Figure 11. The relationship between FS and $h(N - 1)/H$ in the conditions with $u/H = 0.1$.

7. Conclusions

Ribbed smoked sheet (RSS) was studied as a geogrid reinforcement in low CBR lateritic soil for a road embankment. The result from the laboratory test, together with the numerical simulation, confirmed the performance of the RSS reinforcing layer in the road embankments. The RSS functional layer could improve the CBR of the subbase lateritic soil layer. Using 2D FEM simulations, the RSS with two layers installed at $0.1H$ below the top of the road embankment and $0.4H$ below the first RSS layer demonstrated the highest safety factor value among all 61 analysis conditions. To be used as a reinforcement material in road embankments, the RSS is strongly recommended for low CBR soil stabilization. This is because the RSS can act as a weak layer in between the high-CBR soil layer of the road embankment.

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