Geosynthetics Application in Indonesia – A Case Histories

Tjie-Liong GOUW[1]
[1]*Universitas KatolikParahyangan, Bandung, Indonesia*
E-mail: gtloffice@gmail.com

ABSTRACT: The first application of geosynthetics technology was back in 1983, where a high strength geotextile of 200 kN/m was laid to help stabilize the highway built on swampy land toward Soekarno Hatta airport, the gateway to Indonesia. Since then, geosynthetics have been gaining popularity in solving challenging ground conditions for civil engineering development, e.g. stabilization of road development over peat deposits, accelerating consolidation of soft clay, stabilization of foundation over expansive clays, slope stabilization over clay shales formation, retaining walls, ponds lining, breakwater, shore protection and river bank stabilization, etc. This paper presents the author experiences in applying geosynthetics technology in building geotechnical construction over difficult ground condition such as peat, soft clay, expansive soils, and clay shales. It also presents the application of geosynthetics tubes (geotubes) to build containment dykes over soft marine clays.

KEYWORDS: Geosynthetic Reinforcement, Vacuum, MSE wall, Geotubes, Peat, Soft Clay, Expansive Soils, Clay Shales, *Tanah Merah*.

1. INTRODUCTION

Indonesia, apart from being located in an area with high potential seismic hazard as shown Figure 1, it also has various challenging ground conditions, such as: highly compressible peats and soft clay deposits, loose sands prone to liquefaction, expansive clay that induce cracks to roads, housing and low rise buildings, clay shales that cause many slope stability problems, erosion and abrasion that needs erosion control and breakwater system, hilly terrain that often require high slopes and retaining walls to be built, and some other geotechnical issues. Peat and soft clays are commonly found in Java, Sumatra, Kalimantan and Papua (Figure 2). Expansive clays are found in West and East Java particularly in Cikarang and Surabaya area. Clay shales formation are found in Java and Sulawesi. Except in Kalimantan island which is recorded as the only main island with low seismic activity, loose sands that prone to liquefaction and lateral spreading are common along the coastal area of all the islands. Fragmented rocks formation also quite common in all islands.

Ever since its first application in 1983 where high strength geotextiles of 200 kN/m were laid as base stabilization over very soft organic clay deposits for a highway linking Jakarta international airport to the city of Jakarta, geosynthetics technology has been gaining popularity in solving many of the above mentioned geotechnical challenges. e.g. geosynthetics for base stabilization of road embankments, geosynthetics reinforced soils as slope stabilization (Gouw, 1990a; Gouw, 1990b; Lelli et al, 2016; Gouw et al, 2016; Mochtar, 2016; Lelli et al, 2017), the application of prefabricated vertical drain (PVD) in mitigating soft soils settlement (Gouw, 1992; Gouw, 1995;Gouw, 2014), geotextiles tubes as containment dykes and breakwater structures (Saputra and Suhendra, 2016; Hidayat et al, 2017), and many other applications.

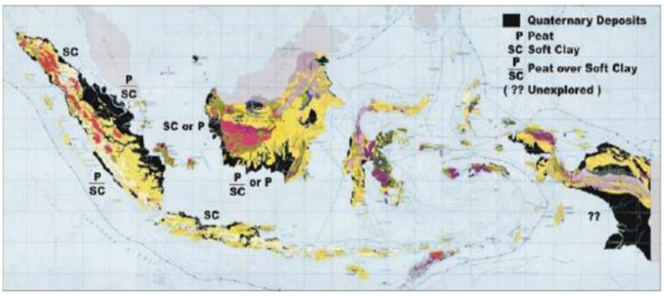

Figure 2 Distribution of peats and soft clays in Indonesia (Cox, 1970; Indonesian Public Work Department, 2002).

This paper discusses the author experiences in applying geosynthetics technology in building road embankments over peat soils, settlement mitigation of soft clay, road and housing foundation over expansive soils, slopes stabilization over soft clay and clay shales, and containment dykes over soft marine clays.

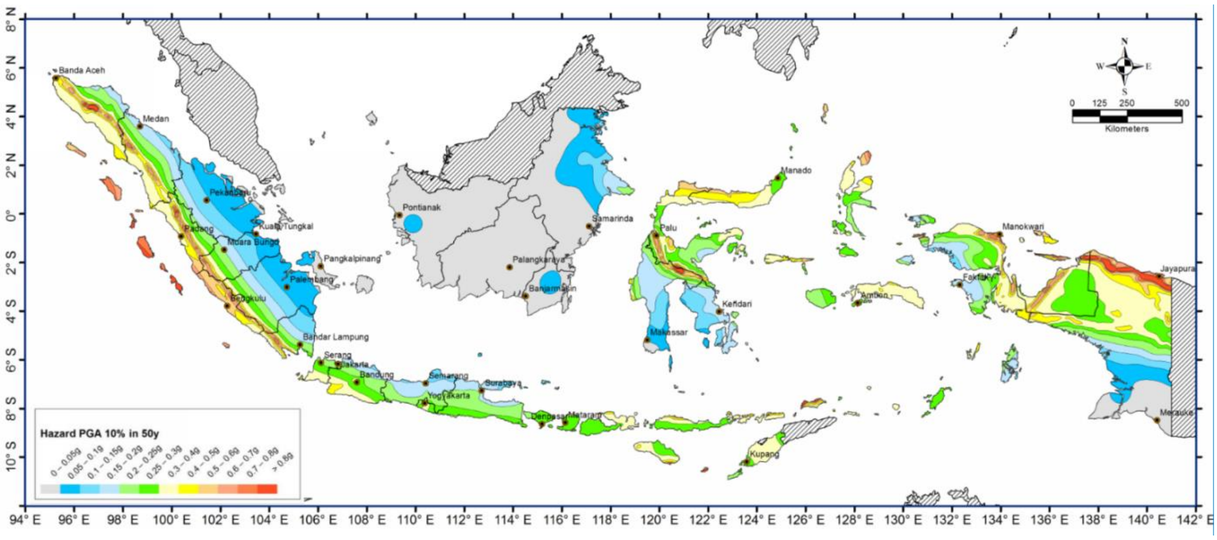

Figure 1 Indonesia Earthquake Map - 500 years Return Period (Hadimuljono, 2017)

2. ROAD EMBANKMENT OVER PEATS

Many of the low land area along the shorelines of Sumatra and Kalimantan islands compose of peat swamps with water content ranging from 250% to 400% sometime can be as high as 600% (Figure 3). Apart from its low bearing capacity, the prime issue is its high compressibility requiring high volumes of backfill material.

Figure 3 Highly compressible peat soils

Traditionally, for road embankment, to counter these geotechnical issues, network of timber trunks is laid, stacked and tied together on top of the highly compressible peat deposits to form a kind of mat foundations. This system is locally known as corduroy foundation (Figure 4). Depending on the compressibility of the peat soils, sometimes it can consist of up to 5 layers of timber trunks. Basically, this corduroy system relies on the tensile strength and the buoyancy effect of the timber trunk when placed under water. Typically, the timber trunks have the following properties: density 4-5 kN/m^3, shear strength 500-800 kPa, tensile strength 4500-6000 kPa, and modulus of elasticity (6~8) x10^6 kPa. However, as this system requires many trees must be sacrificed, environmental awareness made this option unacceptable.

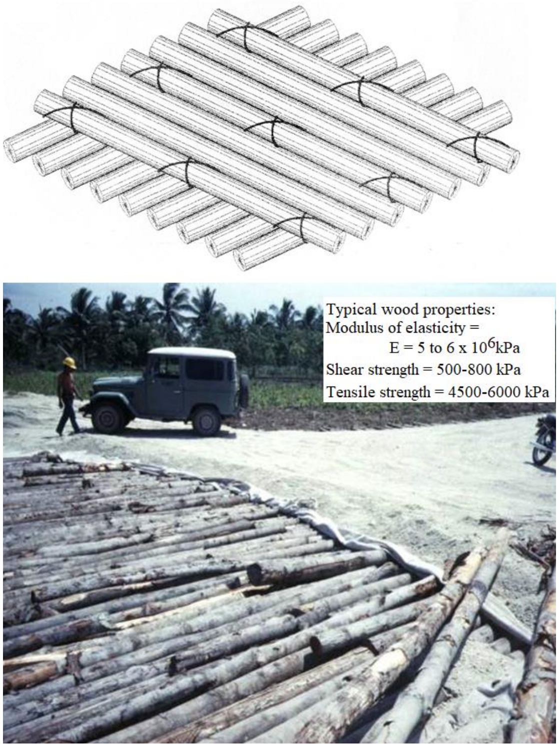

Figure 4 Wooden corduroy system to form timber mat foundation

The other viable solution is using geosynthetics with the aim to keep the road embankment backfill as thin and as light as possible. The construction is started by excavating the peat along both sides of the proposed road alignment, the excavated material is then placed on the proposed road embankment. The intention is to bring the peaty ground to above water level, to form a working platform and to make road side drains (Figure 5). Geogrid and/or geotextile are then laid over the low embankment made of peat. Combination of geogrid and geotextile are often used. Geogrid to provide stability of the road embankment. Non-woven geotextile to separate peat soil with good compacted backfill material. On top of the geogrid the body of the road is then built (Figures 6 and 7).As it is quite difficult to obtain undisturbed samples of peat for consolidation and strength test, the related design parameters are commonly estimated by correlations available in the published literatures, e.g. Mesri, 1973; Mesri and Ajlouni, 2007; Duraisamy et al, 2007; and Bujang at al, 2014. The primary compression index, C_c, generally falls in the range of 1.5 to 5. Secondary compression index, $C_{\alpha'} = C_\alpha/(1+e_p) \approx 2 - 13$. And shear strength values of c' = 0 kPa and $\phi' = 27\text{-}30^o$ are commonly adopted. Considering the wide range of the soil properties, after the thickness of backfill and type of geosynthetics is calculated, trial constructions with zero to two layers of geosynthetics are frequently carried out.

Figure 5 Excavating along both sides of road alignment and forming low peat embankment

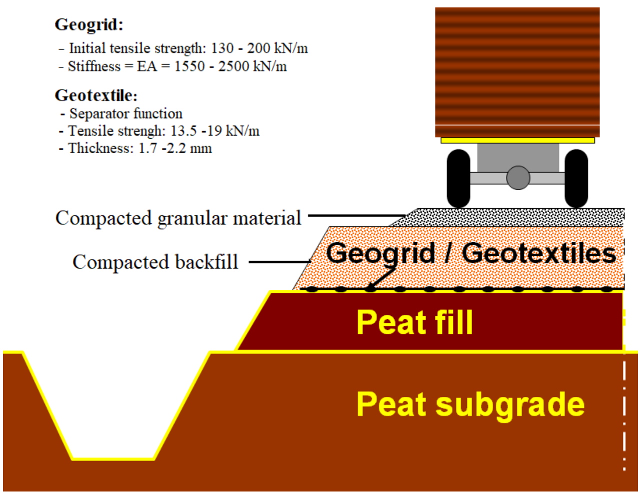

Figure 6 Construction of road embankment over peat swamps (Courtesy of PT Multibangun Rekatama Patria).

Figure 7 Geosynthetic reinforced road embankment over peat swamps

The settlement and the rut depths are measured, and the best solution is adopted. Figure 8 shows the finished road supported by one layer of geogrid and geotextile, with a 65 cm thick base and sub-base construction. It is successfully put into operation and subjected to high frequency of equivalent single axle load vehicles. This type of unpaved road generally requires regular maintenance, the variability of the soil along the road can cause uneven settlement and eventually develop long wavelength differential settlement. Therefore, from time to time it needs to be regraded and re-compacted (Figure 9).

Figure 8 Finished geosynthetic reinforced unpaved road construction over peat swamps.

Figure 9 Regrading and re-compaction required from time to time.

3. SETTLEMENT MITIGATION OVER SOFT CLAYS

3.1 Preloading with Prefabricated Vertical Drains

Owing to the low shear strength and high compressibility of soft clayey soils, a structure cannot be directly built on top of such deposit, as it will suffer intolerable settlement or even bearing capacity failure. In these clayey soils, improvement is normally performed by pre-compression or pre-loading technique, i.e. applying a preload to the compressible soil until a certain degree of consolidation is achieved. The preloading time is the governing factor and very often, a surcharge load, i.e. loads more than the final design load, is applied to accelerate the consolidation process. To accelerate the consolidation further, prefabricated vertical drains (PVD) may be used. Since its first application in 1984 at Balikpapan, East Kalimantan, this technique has practically become the normal practice in Indonesia for accelerating consolidation of soft clay. Most them were applied on reclamation area or in marine clay environment. The case history below discusses the application of PVD to improve 20-32m soft clay found at the height around 660 m above sea level.

Even though Bandung city in West Java is surrounded by mountainous region, a flat terrain at the height of 660-668m above sea level is found to be underlain by 20 to 32 m soft clay deposit. The area is known as Gedebage area. Based on the geological map reported by Bandung Directorate of Environmental and Geology (1990) this Gedebage was once an old vast lake formed at Bandung basin. Desiani and Rahardjo (2017) reported the 20-32m soft clay deposit was the result of Tangkuban Perahu volcanic eruptions, situated some 25 km to the north of the Gedebage area.

In line with the expansion of Bandung city development, thousands of two story houses are being developed at Gedebage area. To avoid future intolerable settlement due to the existence of very soft to soft clay, preloading with PVD was adopted to accelerate the consolidation process. In one of the cluster, the soft clay was found to be around 27m thick (Figure 10).

The ground water table was practically located at the existing ground surface. The properties of the soft clay were found as follows:

- Plastic limit, Wp = 40 - 60%
- Liquid limit, W_L= 80 - 120%
- Natural water content, W_n = 280% near ground surface and gradually reduced to around 80% at 26m depth.
- Void ratio, e_o = 6.0 (at 0 -18 m depth)
  4.5 (18-27m depth)
- Field vane undrained shear strength, S_u = 12 – 18 kPa
- Effective cohesion, c' = 8 kPa
- Effective internal angle of friction, $\phi' = 24^o$
- Coefficient of consolidation, Cv = 0.0073 m^2/day
- Compression index, C_c = 2.3 (at 0 -18 m depth)
  = 1.9 (18-27m depth)

PVD were installed up to 27m depth with equi-triangular spacing of 1.2m and a preload of 5.8m was applied. Figure 11 shows one of the typical settlement record together with the height of the preloading. Asaoka's method evaluation in determining degree of consolidation achieved is presented in Figure 12. Based on Asaoka's method, at the end of 191 days or 97 days after full preloading, the degree of consolidation achieved was 97%. At 192-day, three cone penetration tests (CPT) were conducted. The results were plotted and compared with the pre-treatment CPT as presented in Figure 13. It is clearly seen that the cone resistance increases significantly. By local correlation where a cone factor, N_k, of 25 is used, the undrained strength increased to around 40 - 140 kPa at 0 to 13 m depth, measured from original ground level (OGL), and below 13 m depth it shows even higher values.

Even though Asaoka's method showed the settlement had achieve 97% degree of consolidation and the CPT test also showed significant increase of cone resistance, the settlement record presented in Figure 11 has not yet reached asymptotic state. It was judged that secondary compression was still going on, therefore, the consultant suggested to delay the surcharging removal until the construction of houses started which was scheduled to start one month later. Unfortunately, the monitoring contract was ended and therefore no further settlement data was available.

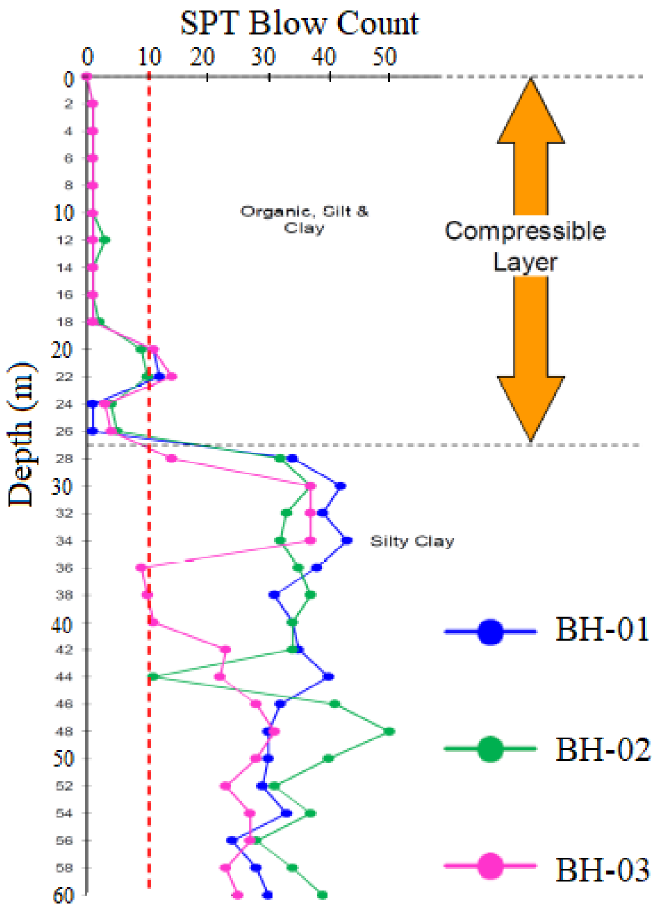

Figure 10 SPT blow count vs depth.

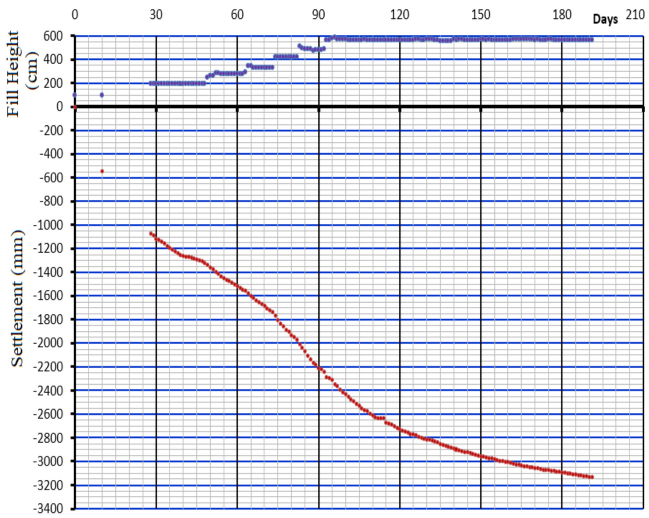

Figure 11 Gedebage typical settlement record.

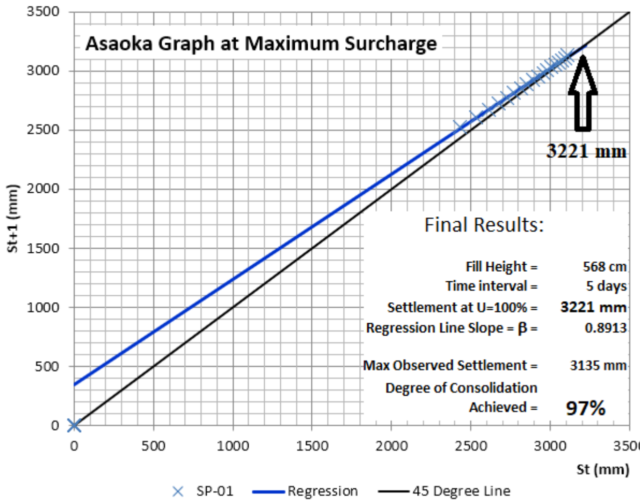

Figure 12 Gedebage – Asaoka's Method.

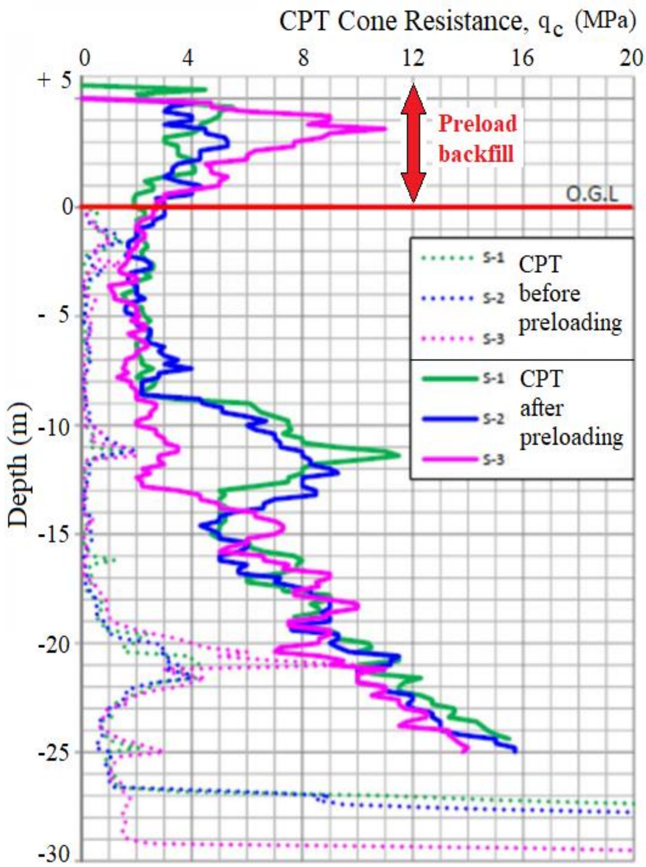

Figure 13 Gedebage – Pre and Post Treatment CPT.

3.2 Vacuum Preloading

Vacuum preloading method utilized atmospheric pressure as surcharge load to accelerate soil consolidation. Its principle is presented in Figure 14. Atmospheric pressure will act pressurizing the soft soils when vacuum is imposed within the soil body. The vacuum pressure within the soil body is created by pumping through an interconnected network of PVD (prefabricated vertical drain), horizontal filter pipes and sand blanket, forming a complete path for spreading the vacuum pressure and facilitating water flow. To be effective an airtight geomembrane cover is required. Figure 15 shows the whole configuration of the vacuum network. When there are sand lenses, vertical slurry wall may be required to cut off the continuous sand lenses or else the vacuum may not work. The system can induce a vacuum pressure of around 80 kPa, with unit weight of soil surcharge of 17kN/m^3, it is equivalent to 4.7m high of surcharging material. The advantages of vacuum preloading over surcharging system are: shorter construction time, lesser earth moving equipment required, shorter consolidation time, and since the consolidating soil layer is subjected to isotropic stresses, it precluded slope stability problem.

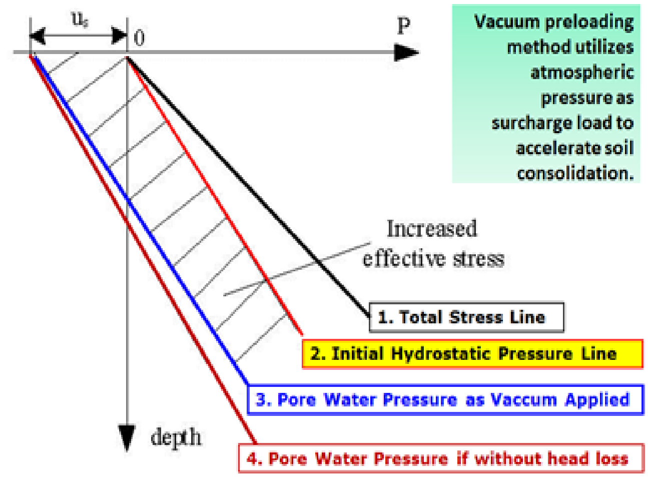

Figure 14 Principle of vacuum preloading(Gouw and Liu, 2013).

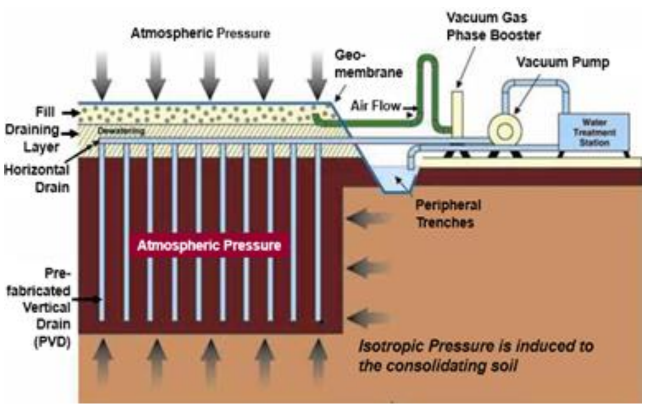

Figure 15 Configuration of the vacuum network (Masse et al, 2001).

This improvement system was put into trial at Gedebage, nearby the area presented in previous case history. The thickness of the soft clay in this vacuum area was around 21m, therefore, the vacuum preloading was carried out with 20 m long PVDs with 1.2m triangular spacing. Figure 16 shows typical vacuum pressure, time settlement and piezometer readings.

Unlike normal practices where vacuum pressure of around 80 kPa can be achieved, at this project the vacuum pressure achieved was only in the order of 74 kPa. Located in a relatively high ground, Bandung area has an average atmospheric pressure of 93.8 kPa. With an effective vacuum pump of 80%, the effective vacuum pressure that can be exerted into the ground shall be around 80% x 93.8 kPa = 75 kPa. Therefore, it was concluded that the vacuum pressure of 74 kPa was acceptable. Apart from this lower vacuum pressure than normal, the vacuum data showed that it took 42 days to achieve 74 kPa. This rather long duration to achieve maximum vacuum pressure might be due to the existence of thin sand lenses at around 2.3-3m depths (normally it takes 14-21 days to achieve 89% atmospheric pressure).

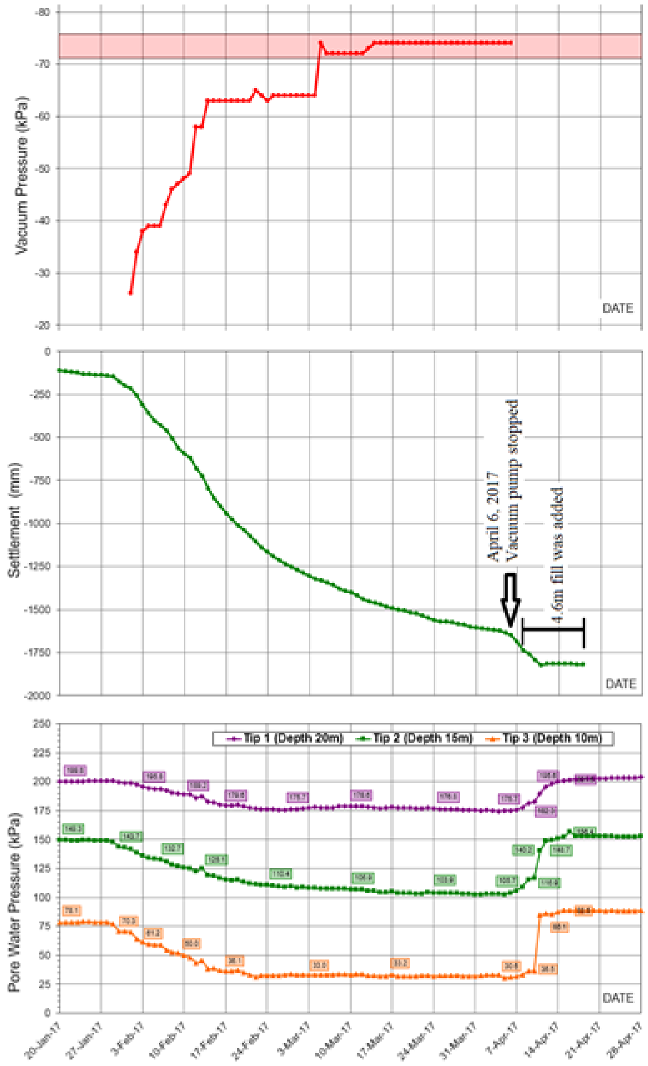

Figure 16 Gedebagevacuum preloading monitoring data.

Based on Asaoka's method, on April 6, 2017, 65 days after the application of vacuum or only 24 days after the soil was subjected to a stable vacuum pressure of 74 kPa, it was found that 94% degree of consolidation has been achieved (Figure 17). Therefore, the project director instructed the vacuum pump to be stopped. The decision was taken without considering piezometer data. Evaluation of the pore water pressure data gave only 80% degree of consolidation achieved (Figure 18). Judging from the fact that re-applying vacuum may take another 40+ days to achieve 74 kPa, to make sure 90% of consolidation was achieved, it was decided to apply soil surcharging of 4.6m high.

Figure 19 shows the comparison of pre and post vacuum undrained shear strength derived from CPT tests. The post vacuum CPT test was performed on April 16, 2017, the undrained shear strength increases by about 3 to 4 times of its original values. Even though vacuum application was stopped a bit too early, compared to conventional preloading system applied at the same Gedebage project area, the vacuum system indeed needs lesser consolidation time and the post treatment undrained shear strength also increases significantly.

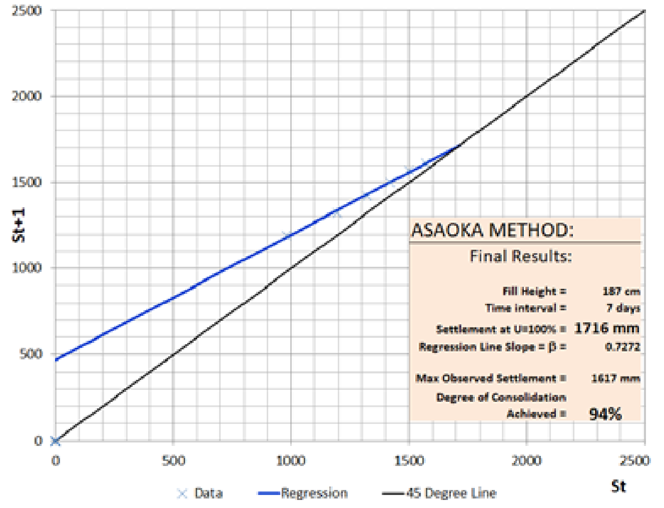

Figure 17 Gedebage vacuum final degree of consolidation achieved.

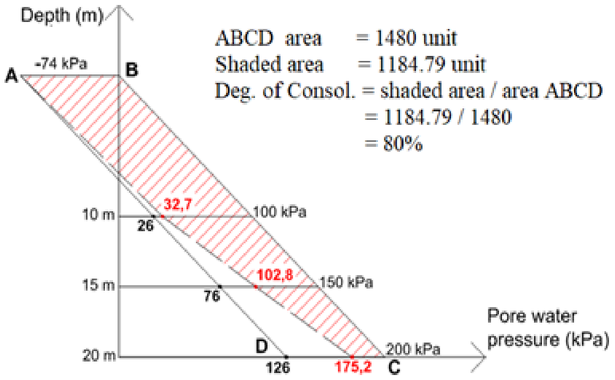

Figure 18 Degree of consolidation based on piezometer data.

4. FOUNDATION OVER EXPANSIVE SOILS

Another large two stories housing complex was being built at Karawangregency, some 80 km east of Jakarta. The typical underlying soil profile is shown in Figure 20.As other area to the east of Jakarta, i.e. Bekasi and Cikarang, the geotechnical problem at this Karawang regency is the clay deposits is expansive. The top 5m to 7m thick of the expansive clay has the following characteristics,

- Bulk unit weight, γ = 15– 17 kN/m^3
- Void ratio, e_o = 1.09 – 1.87
- Atterberg limits, PL = 22-60% ; LL = 51-105%; SL = 17-34%
- Activity = 2.8 – 4.5
- Natural water content, W_n = 34 - 67%
- Shear strength: c' = 30-45 kPa and ϕ' = 20-31^o
- Compression index: c_c = 0.20 – 0.48
- Swelling potential, S_p = 3.5 %
- Swelling pressure, σ_{vs}= 90 kPa
- Permanent groundwater level = 3-4m beloworiginal level

Previous record showed that roads and houses built on those areas without any knowledge on the existence of expansive soil were experiencing severe cracks at the change of dry to wet season as shown in Figure 21 and 22.

To mitigate the clay expansion that would induce damages to the road construction, combination of soil replacement, counter weightand lining were adopted as shown in Figure 23. The top 1m of

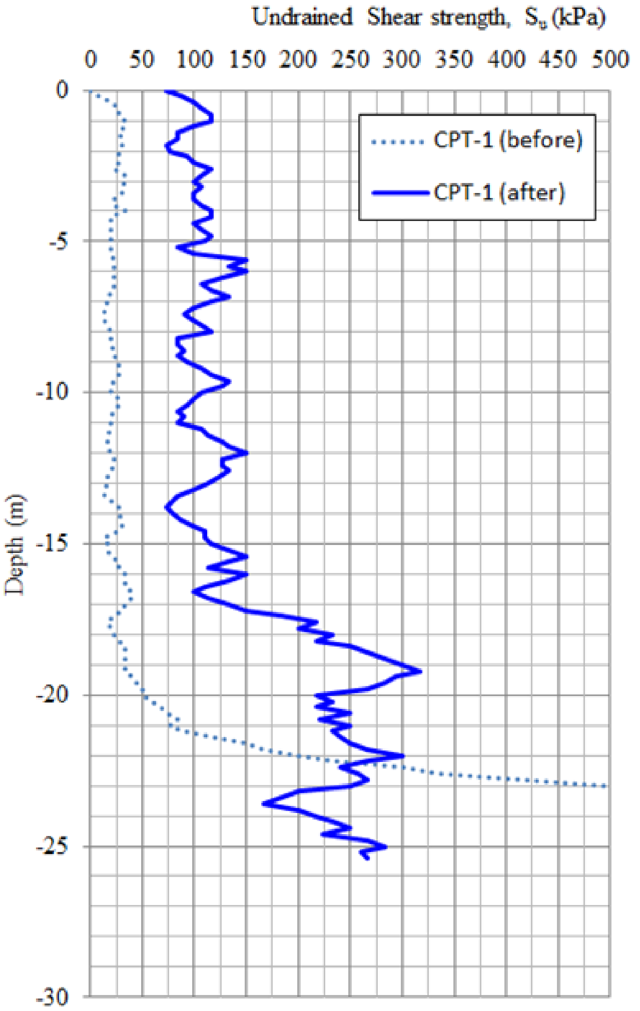

Figure 19 Gedebage vacuum - pre and post vacuum undrained shear strength.

expansive clay was removed and replaced with limestone fully covered by non-woven geotextile. To prevent extreme water content changes the top and the sides were covered by geomembrane up to the level of permanent groundwater. Since the project was located at low area, its elevation needed to be raised by 1.20m, for this 1.20m thick backfill under the road, it was constructed by 50cm compacted non-expansive clay with the intension to reduce surface water migration and 70cm compacted granular backfill. Both backfill materials and limestone cover acted as counter weight against the remaining 2-3m thick expansive clay susceptible to water inundation. Proper drainage system is provided along both sides of the road to drain out rain water.

For two storey housing foundation, a suspended slab sitting on strip footing system shown in Figure 24 was adopted. A gap of 8cm below the ground floor was provided so that when the soil beneath undergo heaving, it will not exert high pressure on the suspended slab. The maximum thickness of expansive clay above the permanent water level allowed was 4m. The counter weight provided was 1m of limestone and 50cm of compacted non-expansive clay. Non-woven geotextiles were laid sandwiching the limestone to act as separator between the soil layers. Drainage around the houses were designed properly so that it would not flow back underneath the houses.

Both solutions were found to be satisfactory and no cracks or undue damages were observed after one year of construction. The design calculation was based on the formulas given by Nelson et al (2015) as presented below:

$$\rho = C_H\, H \log\left[\sigma_{cv}/\sigma_f\right] \qquad (1)$$

$$C_H = \frac{\varepsilon_s}{\log\left(\sigma_{cv}/\sigma_i\right)} \qquad (2)$$

where ρ = total free field heave ; C_H = heave index; H = thickness of expansive clay susceptible to water inundation; σ_{cv} = vertical pressure to prevent swelling obtained from constant volume oedometer test; σ_f = final vertical pressure acting on top of expansive clay; ε_s = swelling potential from free swell oedometer test; σ_i = vertical pressure at which the sample is inundated; and σ_{cs} = vertical pressure to push back swelling back to zero obtained from free swell oedometer test.

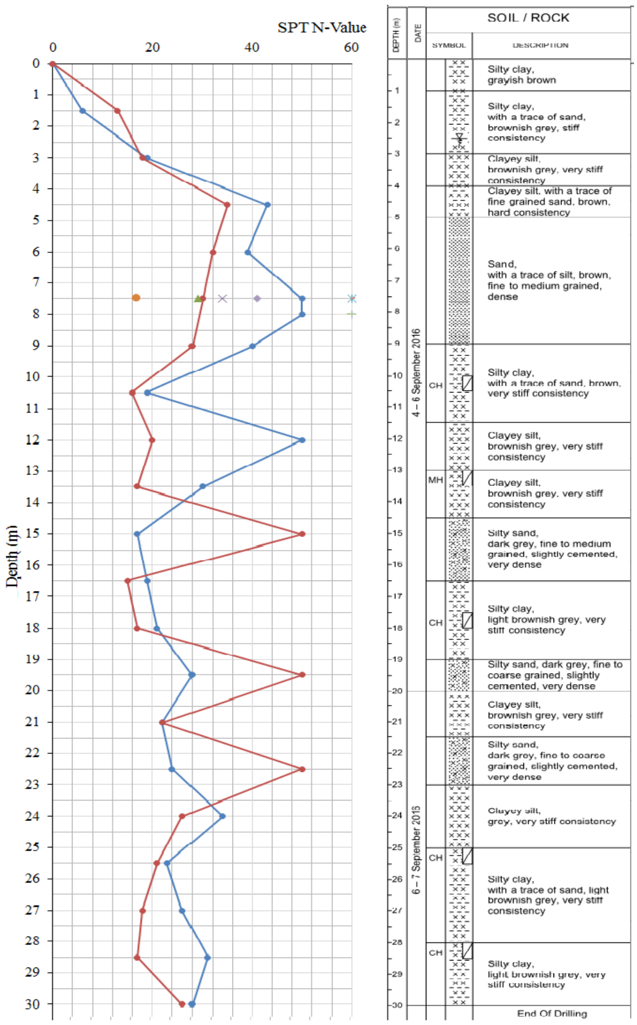

Figure 20 Karawang housing complex - typical soil profile.

Figure 21 Cracks on roads due to expansive soils.

Figure 22 Cracks on houses due to expansive soils.

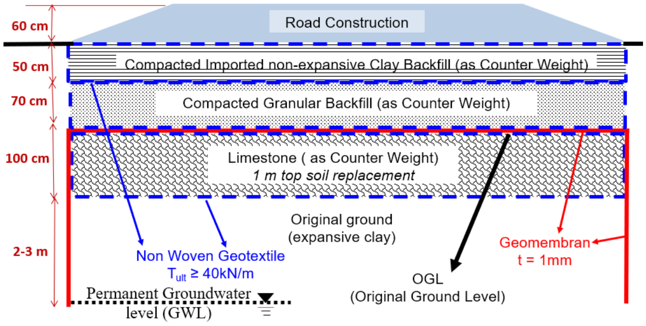

Figure 23 Counter weight and lining system for road foundation sitting on expansive clay.

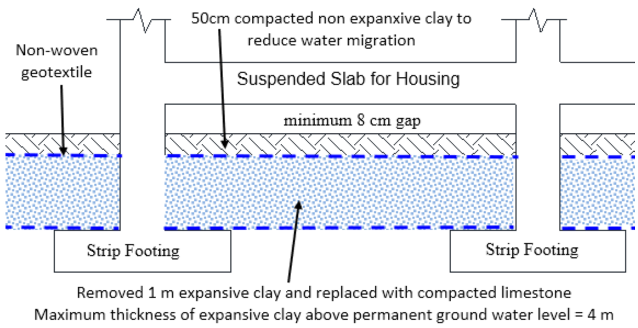

Figure 24 Suspended slab for housing on expansive clay.

5. SLOPES STABILIZATION

5.1 Geosynthetics Reinforced Embankment over Soft Clay

In 2017 a new railway linking downtown Jakarta to Soekarno-Hatta, the international airport of Jakarta was built. To avoid interferences with road traffics, several bridges and culverts were built along this new12 km long Ba-Soetta Rail. Almost half of it had to be built on soil embankment. The highest embankment section was 13m and it crossed paddy fields characterized by soft soil. To curtail the needs of large land acquisition, the project owner set the requirement to build the railway embankment slopes as steep as possible. This 13m high embankment was underlain by clay and silty clay with soft consistency at the top 10m followed by stiff consistency at greater depth. The groundwater table varies between -2.5m from the ground surface up to almost at the surface level.

The initial proposed construction was a combination of reinforced cantilever walls sitting on a driven piles foundation containing the embankment body which was reinforced with layers of woven geotextile wrapped around right at the back of the RC walls. This solution was found to be costly. Finally, a back-to-back mechanically stabilized earth (MSE) embankment coupled with a foundation improvement by means of soft soil replacement and basal reinforcement presented in Figure 25 was adopted.

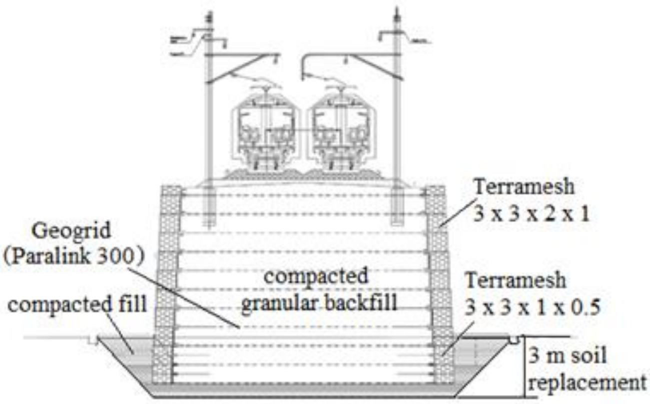

Figure 25 Ba-Soettarail embankment constructed with back to back MSE wall and soil replacement.

The back to back MSE embankment are basically geosynthetic reinforced soil structures with a slope inclination of not less than 70 degrees. By placing geogrid reinforcing elements within compacted granular backfill, from one end of the wall to the other end, and tied it to the wall facings, a near vertical soil retaining system can be built. This geosynthetics reinforcement system resulting in an overall project saving of 40%.

Compared to RC retaining walls, MSE walls can tolerate larger total and differential deformation. However, a MSE wall with precast concrete panels facing cannot tolerate as much deformation as welded or double twisted wire mesh gabion facing because of its potential damage to the precast panels and unsightly face panel separation.

Therefore, flexible welded wire gabion facings combined with the geogrids reinforcement (known as Terramesh system) shown in Figure 25 were adopted. The tensile element used in this project was high strength polyester geogrids coated with LLDPE for the best long-term performances and have an ultimate tensile strength equal to 300 kN/m (Paralink 300). The geogrids were laid continuously from a side of the embankment to the other side with no mechanical or frictional joints. The geogrids vertical spacings were 1m and the maximum width of the embankment body reached 16 m at the base.

The MSE walls stability and deformation were analysed by limit equilibrium and Plaxis finite element software. Train variable load of 200 kPa was adopted, and a horizontal seismic coefficient, k_h, of 0.3 was considered. The allowable minimum factor of safety under static and seismic condition was 1.7 and 1.2, respectively. In the area previously used as paddy fields, an average of 3m thick soft clay was found. To achieve the required safety factor and to limit deformation, the 3m thick soft clay was replaced with a well compacted granular fill. Before placement of the granular soil, a layer of non-woven geotextile was installed to separate the fine and cohesive existing soil from the granular fill placed on top. In addition to this, a double layer (both in the railway longitudinal and transversal direction) of high strength geogrids was installed directly onto the geotextile to serve as basal reinforcement (Figures 26 - 27).

Figure 26 Removal of the 3m soft clay.

Figure 27 Separator geotextiles and basal geogrids reinforcement.

Selected backfill material for the embankment body was sandy gravel soil, compacted every 25cm thick lift, belonging to the class A-1 or A-3 of AASHTO soil classification system (AASHTO M 145 or ASTM D3282), with a minimum required effective friction angle after compaction equal to $\varphi = 30°$. The construction works was started in January 2017 and by December 2017 the railway had already in trial operation and no problem was encountered. Figures 28 to 30 show the construction process and the completed construction.

Figure 28 Compaction and building up the embankment.

Figure 29 Partly completed anchored gabion facings.

Figure 30 Completed MSE wall construction.

5.2 Reinforced Slopes over Clay Shales

To expand tourism industry at the exotic and breath-taking scenery of Tana Toraja, a new airport with 2 km long and 210 m wide runway is being built. This new airport is situated in hilly terrain; therefore, massive cut and fill earthworks must be undertaken to get a flat surface for the runway. The first stage of the runway construction was undertaken in 2015. It involved the design and the execution of a 100m long and 15 m high retaining structure as runway support. The retaining structure had to be erected in between two hills. Later in a second stage, the retaining structure will be topped with an additional 10 m high structure to reach the final runway elevation. The final height of the retaining structure shall be 25 m. The main geotechnical problems related to the design and the execution of this works were: high seismicity of the area with a PGA=0.3g for a return period of 500 years, and the presence of clay shale foundation soils which easily loss its strength when exposed to atmosphere.

Clay shale is a sedimentary rock originated from clays that become rock due to long term high pressure deep in the ground. Over time geological events brought the clay shale formation to near surface. This clay shales, when dry and undisturbed, are hard and have high shear strength. However, when excavated and exposed to open air they are easily degraded and dramatically loss their shear strength. Gartung (1986) reported unweather clay shale can have an effective cohesion as high as 85 kPa with internal friction angle of 41^o. However, when exposed to open air, it weathered quickly, and its shear strength fell to as low as zero cohesion and friction angle of only 9^o (Figure 31).

Figure 32a show the hard, rock like, clay shale freshly exposed to open air. Figure 32b shows the clay shales disintegrated after just a few days exposed to atmosphere, even if they still look like rock it is easily spalling off and disintegrated by just applying small forces onto it. Based on the known properties, at this project, for the upper 3 m of
the clay shale where the retaining walls were founded (after removing the top 2-3 m of soil layers),the shear strength were reduced to c' =20 kPa and $\phi' = 17^o$.

Seismic design criteria, space limitation and land acquisition issues made unreinforced slopes option unsuitable. Three different types of retaining structures were considered during the planning stages, i.e.: concrete mass gravity walls, bored piles and hybrid reinforced soil slope (HRSS) combining anchored gabion units and high strength geogrids.

Selection criteria given by the owner's geotechnical committee were: the retaining structures should have a very permeable facing in order to rapidly drain rainfall water and to dissipate hydrostatic pressure developed in the cohesive backfilling soil; it should flexible enough to accommodate potential differential settlements and to absorb dynamic shocks in case of a seismic event; must be built in 2 months' time; must maximize the employment of locally available unskilled manpower; and the overall cost must be economical. Based on all the above criteria, Hybrid Reinforced Soil Slope (HRSS) combining anchored gabion units and geogrids was selected as the best suitable solution. The main components of the proposed HRSS are illustrated in Figure 33. The anchored gabion units were made of hexagonal double twisted wire mesh 8x10 Galmac (Zn-Al 5%) polymer coated steel wire diameter 2.7/3.7mm. Geogrids of initial tensile strength of 300 kN/m, made of high tenacity polyester yearns tendons encased in a polyethylene sheath, were used.

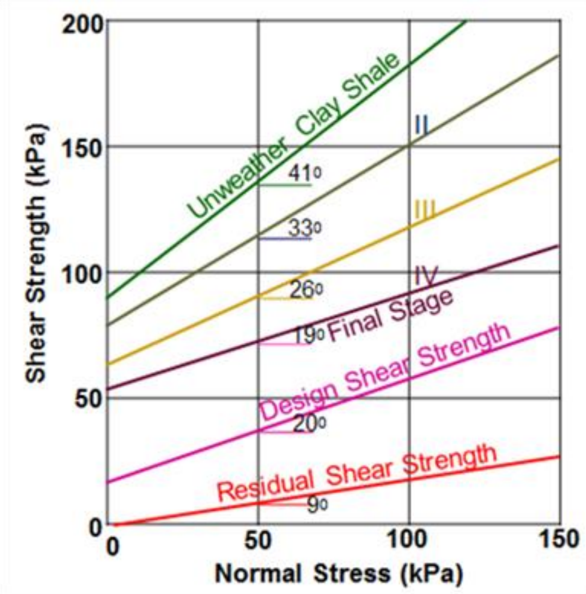

Figure 31 Shear strength degradation of clay shales.

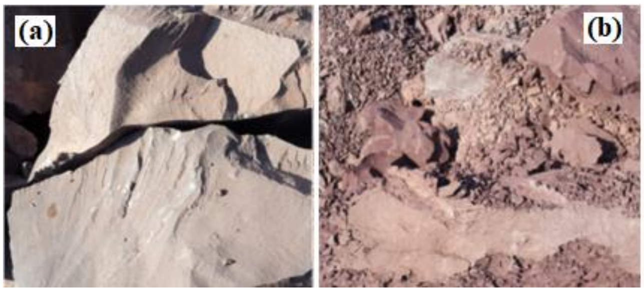

Figure 32(a) Freshly Exposed Clay Shale (b) Disintegrated Clay Shale.

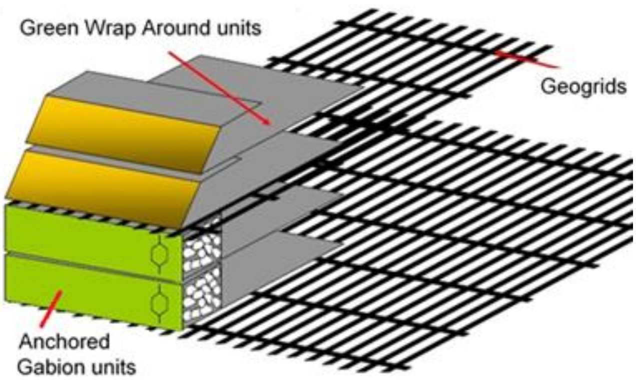

Figure 33 Hybrid reinforced soil structure main components.

The first stage retaining structure height was equal to 15m, but it had to be designed for a final target height of 25m, the subsequent additional 10m shall be constructed on the next budgeting year. One of the HRSS typical section is illustrated in Figure 34. The 25m high structure was distributed in 5mhigh stepped berms. To make a stiffer base, the first two berms, starting from the foundation level,was built using double twist anchored gabion units. Meanwhile, for the upper 3 berms, Green Wrap Around units with 60^{o}inclination to horizontal were used. The primary reinforcements were geogrids having an ultimate tensile strength equal to 300 kN/m with an average vertical spacing equal to 1.0 m. The geogrid length ranged from a minimum of 5 m at the top to a maximum of 25 m at the base.

The design was first carried out using a Limit Equilibrium Method based software. Since no information regarding structure deformation and settlements amount could be provided by this LEM software, the HRSS was checked also using FEM software PLAXIS 2D. Figure 35 shows the input parameters used for the FEM analysis. A pseudo-static model was used to investigate the behaviour of the structure under a seismic event causing an additional horizontal mass acceleration equivalent to half of the PGA i.e. equal to k_h = 0.15 g. Both the LEM and FEM analysis gave comparable and acceptable factor of safety required by the project (Table 1).

Since the area had high rainfall intensity, the effect of time-dependent variation of precipitation on the stability of HRSS was carried out by using Plaxis 2D FEM software which can perform transient groundwater flow analysis. To minimize rainwater seepage to the HRSS that can affect its stability, a free draining material covered with light non-woven geotextile was provided behind the HRSS to catch the seeping groundwater from the surrounding area and then draining it outside the HRSS through sub-drainage system provided below the base at certain intervals (Figure 34). Surface top soil and behind the gabion of HRSS were protected with low permeability material, e.g. geosynthetic clay liner.

Figure 36 and 37 respectively shows the predicted deformation under static and seismic condition. Based on the design, the actual construction was carried out in December 2015 and finished in January 2016. To avoid degradation of the clay shale, the foundation of the HRSS wall was constructed in 8m strip and in fastest construction time possible. Despite several issues related to the project, the first stage of the Tana Toraja airport runway construction has been effectively completed within the required two months' time frame.

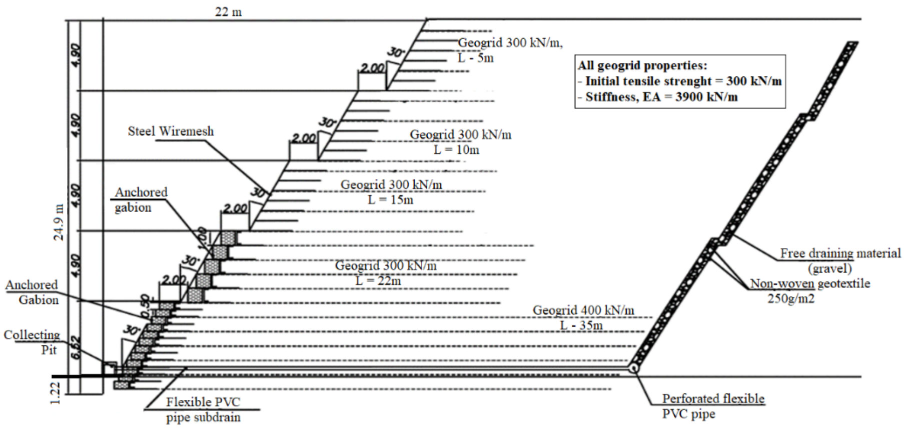

Figure 34 Typical HRSS cross section for Tana Torajaairport retaining wall.

SOIL DATA INPUT

Mohr-Coulomb Soil Model

ID	Name	Type	γ_{unsat} [kN/m^3]	γ_{sat} [kN/m^3]	ν [-]	E_{ref} [kN/m^2]	c_{ref} [kN/m^2]	ϕ [°]	ψ [°]
1	Structural Soil	Drained	17.0	17.0	0.30	40000.0	42.0	16.0	0.0
2	Tator-Clay Shale-0	Drained	17.0	17.0	0.30	10000.0	20.0	17.0	0.0
3	Tator-Clay Shale-1	Drained	17.0	17.0	0.30	10000.0	20.0	24.0	0.0
4	Tator-Clay Shale-2	Drained	18.0	18.0	0.30	20000.0	20.0	30.0	0.0
5	Tator-Clay Shale-3	Drained	19.0	19.0	0.35	20000.0	20.0	32.0	0.0
6	Tator-Clay Shale-4	Drained	20.0	20.0	0.35	1E+05	40.0	37.0	5.0

GABION

Linear Elastic Model

ID	Name	Type	γ_{unsat} [kN/m^3]	γ_{sat} [kN/m^3]	ν [-]	E_{ref} [kN/m^2]	E_{incr} [kN/m^2]	y_{ref} [°]	R_{inter} [°]
7	Gabion Filling	Drained	17.5	17.5	0.35	40000.0	0.0	0.0	1.00

GEOGRIDS

ID	Name	Type	EA [kN/m]	N_p [kN/m]
1	geogrid 300 kN/m	Plastic	3.9E+03	272.00

Figure 35 FEM input soil parameters for HRSS stability analysis at Tana Toraja Airport.

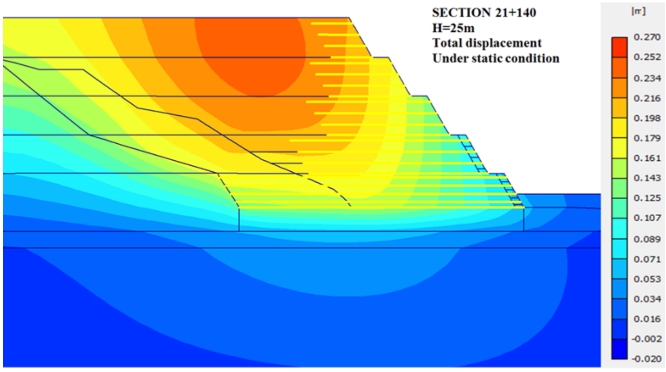

Figure 36 Predicted deformation under static condition.

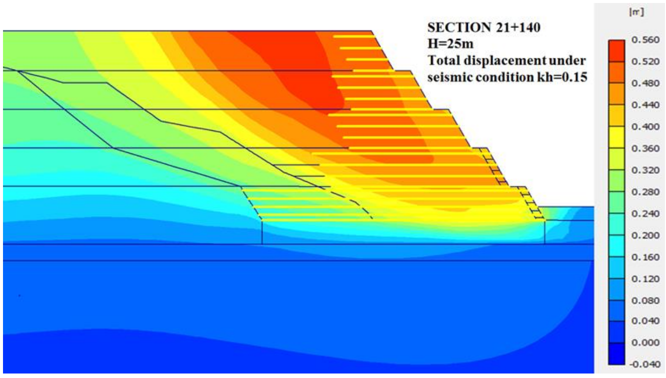

Figure 37 Predicted deformation under seismic condition.

Figure 38 shows part of the construction stages. Figure 39 shows the completed structure and the fully vegetated facing of the reinforced soil slope few months after the completion of the works.

Figure 38 Laying the high strength geogrid and backfilling.

Figure 39 Completed HRSS wall with vegetation on the wall face.

6. CONTAINMENT DYKES OVER SOFT MARINE CLAY

The expansion of a fertilizer factory in Bontang, East Kalimantan, needs to carry out land reclamation over a swampy tidal land. The area located adjacent to a conserve mangrove forest. No damages to the mangroves shall be allowed during the land reclamation process. The sea bed of the proposed reclaimed area is about 2 m below the mean sea level. The required height of the finished reclaimed land is at least 1.5 m above the highest sea water level.

Soil investigation reveals the upper 15m of the seabed compose of very soft to soft clay layer with the following characteristics:

- SPT blow counts = 2 – 6 blows/ft
- Bulk unit weight, γ = 15 – 16kN/m^3
- Void ratio, e_o = 1.55 – 2.0
- Atterberg limits, PL = 20-30% ; LL = 60-80%, PI = 40-55%
- Liquidity index = 0.7 – 1.0
- Natural water content, W_n = 55–72%
- Undrained shear strength: $S_u$increasing from 5 to 10 kPa
- Compression index: c_c = 0.55 – 0.80

The material used to reclaim the extension area shall be taken from the nearby sea port which will be dredged to provide a deep-water pathway for cargo ships. Soil investigation drilling carried out in the proposed dredging area shows that the sea bed consists of soft clay and sandy material, as such it is anticipated direct dumping to the reclaim area will not be viable as it will cause siltation to the adjacent conserve mangrove forest and eventually may induce damages to the mangroves. The local authority put the requirement that the reclamation can be carried on only if the process can safeguard the conservation area.

Looking into the requirement, the options is either to build sheet pile walls along the perimeter of the proposed reclamation zone or a dyke system. Low sea bed and many mangrove trees at the area would not allow a sheet piling barge to enter the area easily, also sheet piles are very expensive. To build a kind of rock fill dykes is not possible as rocks are not available and need to be important from Sulawesi island which need high transportation cost. Building a conventional dyke by a kind of geotextiles reinforced earth embankment will be difficult as the underlying soil is very soft and hence have extremely low bearing capacity, apart from that placing and contain the material on top of the normal geotextiles sheets will also be difficult, as the wave action can easily wash away the material. Therefore, the best option available is to utilise geosynthetic tube or geotube as shown in Figure 40. With this geosynthetic tubes the sand materials used to fill out the tubes will not be washed away by waves or water currents.

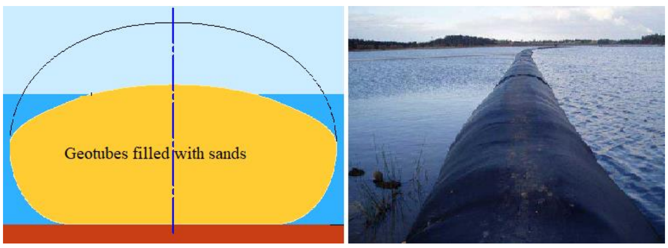

Figure 40 Geotubes used as dyke.

However, direct placement of geotubes on the soft sea bed will cause the geotube settling too much or suffer from bearing capacity failure. Looking into the availability of large number of mangrove tree trunks gathered from the removal of the mangroves during the preparation of the proposed reclaimed area, three layers of mangrove tree trunks are tied together to form a corduroy system placed below the geotubes. Plaxis FEM analysis result shows that with the mangrove trunks corduroy placed at the base and act as the foundation for the geotubes, the system will be stable, and the predicted settlement shall be around 40 cm (Figures 41 and 42). The factor of safety is in the order of 1.1 to 1.3. Note that Kalimantan is the only main island in Indonesia with low seismic activity. Therefore, seismicity is not an issue.

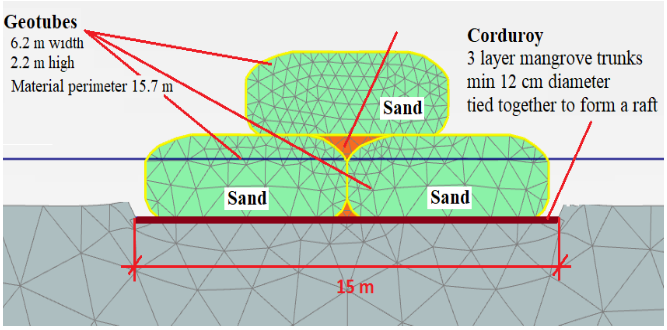

Figure 41 Three layers geotubes sitting on top of corduroy system.

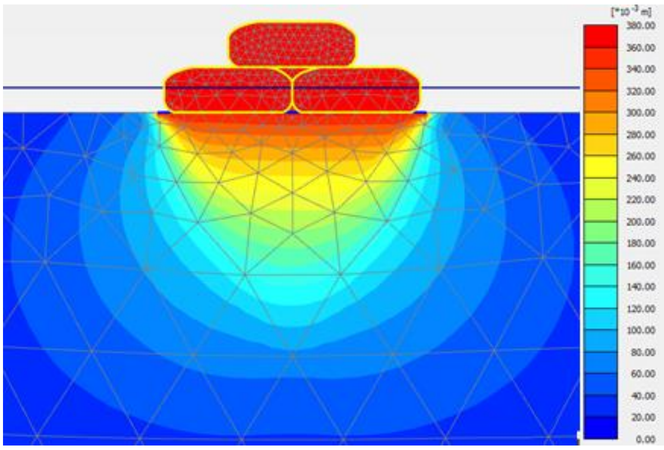

Figure 42 Predicted maximum settlement is 400mm.

Construction with this system is now going on and around 25% completed. Figures 43 to 45 show the corduroy system, the placement of geosynthetic tubes material on top of the corduroy, the completed geotube dyke. The measurement so far shows that the system works as predicted.

Figure 43 The Corduroy made of three layers of mangrove trunks.

Figure 44 Laying of the geotubes on top of the corduroy.

Figure 45 Completed geotube dyke.

7. CONCLUDING REMARK

The paper elaborates how the right application of geosynthetics technology help solving challenging ground conditions in various islands of Indonesia, from building road over peats, mitigating settlement problem of low rise residential building, preventing damages to roads and houses built on expansive soils, constructing stable high embankment for railway over soft clay, building high reinforced slopes over clay shales formation for airport's runway, and building dyke over soft marine clay.

Ever since the first application of geotextile underneath the highway from North Jakarta to Jakarta international airport in 1983, geosynthetics have been widely accepted in Indonesia as one of the alternative in solving geotechnical problems faced by engineers. Some other applications that have been put into practice are landfills, shrimps and fish ponds lining, reinforced pavement, river bank protection, abrasion prevention by concrete mattress, erosion control, roof garden, etc. Further other applications, such as geotextile encased piles, geocells foundation, etc. also being explored.

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