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Earth reinforcement technique with geosynthetics in ASEAN region

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ABSTRACT: The soil reinforcement technique has been used in the ASEAN region for almost 20 years. Over this period of time much experience has been gained with various reinforced soil applications, and the benefits of this technique have been well-proven. However, a number of issues remain regarding the use of this technique in the ASEAN region. The paper describes the various reinforced soil applications used in the ASEAN region, highlights their local history, and discusses the issues that are still to be resolved.

1 INTRODUCTION

When considered in a modern context reinforced soil may be thought of as the reinvention of an old technique. There are numerous examples from antiquity where the reinforced soil technique was used to construct stable earth structures. Some of these structures are still in existence today, e.g. several ancient Ziggurats in Iraq and parts of the Great Wall of China. In the modern context however, the reinforced soil technique really evolved in the 1970's into what it is today. Firstly, the use of metallic reinforcements, and later, polymeric reinforcements has enabled this technique to be used in many geotechnical and structural applications. The range of applications using the reinforced soil technique is shown in Figure 1.

The modern reinforced soil technique first came to the Association of South East Asian Nations (ASEAN) in the early 1980's. This was in the form of proprietary retaining wall systems based on metallic strip reinforcement, and of geosynthetics for the basal reinforcement of embankments over soft foundation soils. Today in ASEAN a number of different reinforced soil techniques are used, namely, basal reinforced embankments over soft foundation soils, basal reinforced fills over super-soft foundations, basal reinforced piled embankments, reinforced fill slopes, and reinforced soil retaining walls. These applications are discussed in more detail below.

2 DESIGN METHODOLOGY USED

The methods of design and analysis that have been developed for reinforced soil structures have been based on either proprietary, generic or hybrid methods. The evolution of these various groups of analysis methods and their establishment in different national design codes has lead to different design approaches being adopted in different countries. Today in ASEAN a number of different design methods for reinforced soil structures are used. These range from well-documented methods in recognised codes of practice, e.g. BS 8006 : 1995, NCMA (1995), to obscure hybrid design methods proposed by specific geosynthetic manufacturers. It is to be recommended that where ever possible engineers use the well-documented methods, as these have a proven record and are based on well-identified and consistent principles.

From a fundamental design viewpoint the forces causing disturbance must be resisted by the forces providing stability with an adequate margin of safety. The various disturbing and resisting components in reinforced soil design are listed in Table 1. The components that affect the magnitude of the disturbing forces are soil self weight, external loads, groundwater effects and seismic events. How these disturbing forces are represented through and around the reinforced soil zone is dependent on the method of analysis used. Changing the method of analysis, and its basic assumptions, can lead to changes in the magnitudes of the distribution of disturbing forces within the reinforced soil zone. Clearly, the appropriate method of analysis used should reflect as closely as possible insitu measurements.

The components that affect the magnitude of the resisting forces are soil shear strength, reinforcement resistance, groundwater effects and deformations. How these resisting forces are represented throughout the reinforced zone is also dependent on the method of analysis used. The appropriate method of analysis

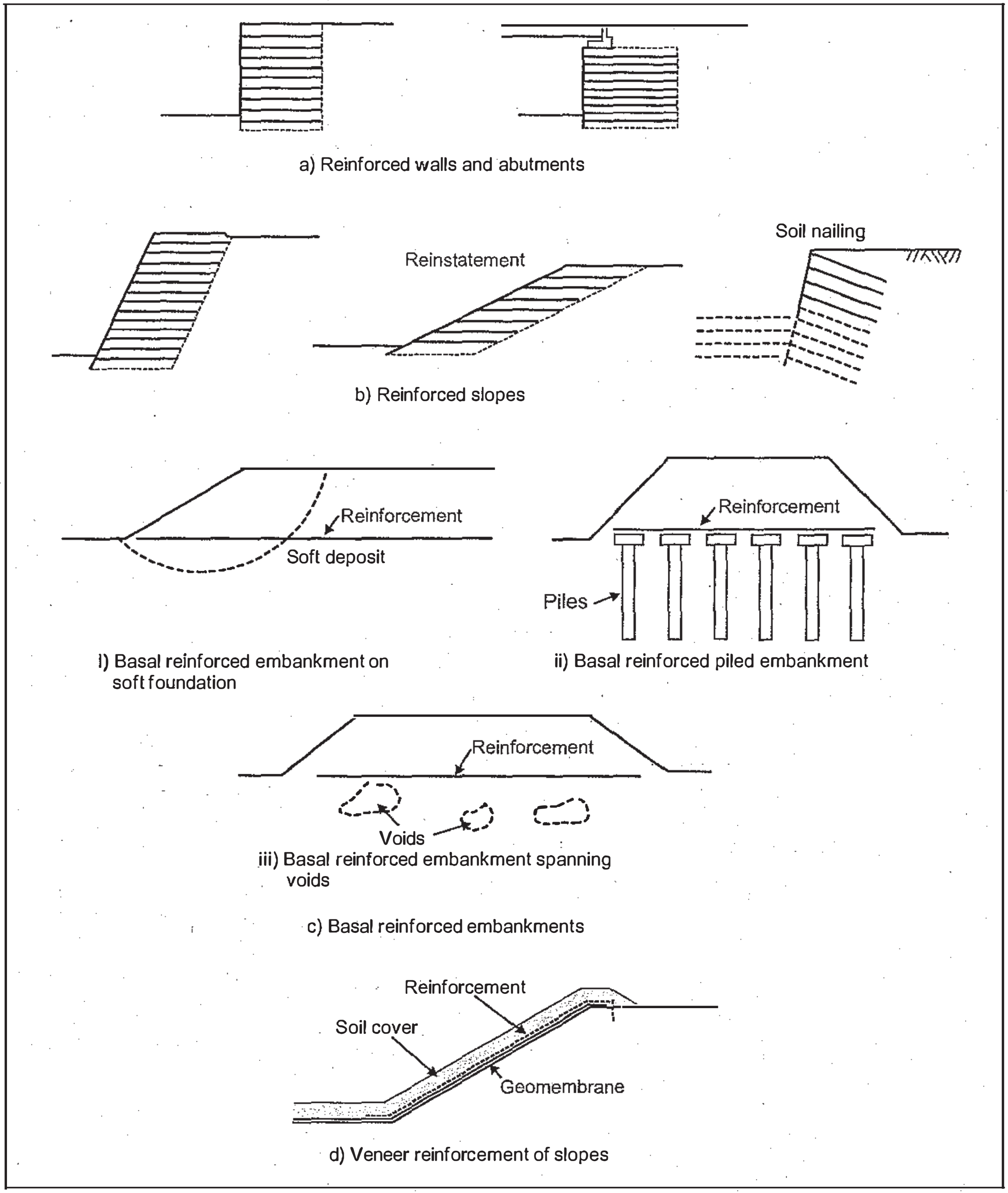

Figure 1. Range of applications where the reinforced soil technique is used.

should also portray the resisting components as accurately as possible.

In deriving the best economies from reinforced soil it is important that the most appropriate method of analysis and design be used in order to accurately model the stresses causing disturbance within the soil mass. At the same time the appropriate method of analysis and design should also model accurately the stresses providing resistance. With soil reinforcement analysis methods still evolving and being refined the technique has still some way to go to reach its maximum economic potential in ASEAN (and elsewhere).

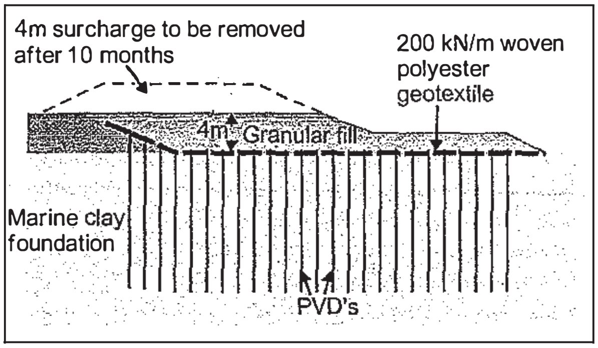

Figure 2. Use of geosynthetic reinforced embankment to construct the depot platform at the Seletar Mass Transit Depot, Singapore.

Table 1. Components affecting the magnitudes of disturbing and resisting forces in reinforced soil design.

Components affecting magnitude of disturbing forces	Components affecting magnitude of resisting forces
Soil weight	Soil shear strength
External loads	Reinforcement resistance
Groundwater effects	Groundwater effects
Seismic events	Deformations
Method of analysis	Method of analysis

3 BASAL REINFORCED EMBANKMENTS

Figure 1c lists the various basal reinforcement techniques in general practice. In ASEAN three techniques are in use. These are geosynthetic reinforced embankments on soft foundations, geosynthetic reinforced fills over super-soft foundation soils and geosynthetic reinforced piled embankments.

3.1 *Geosynthetic reinforced embankments on soft foundations*

Geosynthetic reinforced embankments on soft foundation soils were first used in ASEAN in the early 1980's. Since that time the technique has been used successfully in Indonesia, Singapore, Brunei, Malaysia, Thailand, Philippines and Vietnam. Three of the earliest examples of the use of this technique in the ASEAN region were for highway construction at Chengkareng, Indonesia in 1984, for airport construction at Semarang, Indonesia in 1985, and for the foundation treatment for the Seletar Mass Transit Depot in Singapore in 1985, Figure 2.

The example depicted in Figure 2 involved the widening of an existing earth platform to enable the construction of the Seletar Mass Transit Depot. The foundation consisted of soft marine clay with undrained shear strengths ranging from 10 to 15 kPa. To enable the new earth platform to be completed as soon as possible it was decided to surcharge the new area and use prefabricated vertical drains (PVD) to accelerate consolidation of the soft marine clay. The height of the completed earth platform was 4 m above foundation level and it was determined that an additional 4 m of surcharge for a 10 month period would be required to meet the construction programme for the consolidation of the soft marine clay. To ensure stability of the surcharged fill areas a woven polyester geotextile reinforcement of 200 kN/m tensile strength was used beneath the fill areas.

The most common design procedure used for basal reinforced embankments on soft foundations is the well defined, limit equilibrium approach adopted by such Codes as BS 8006 : 1995. This leads to conservative designs as far as the determination of the tensile properties of the basal geosynthetic reinforcement is concerned, which is just as well as normally the amount of information available on the soft foundation soil is limited.

The types of geosynthetic reinforcements used in basal reinforced embankments over soft foundation soils are either single layers of high strength woven polyester, single layers of high strength woven polypropylene, or, in some instances, multiple layers of geogrids.

While this technique has been used for a considerable period of time in ASEAN, and is relatively common, there are a number of issues that still need to be addressed, namely;

- suitability of different analysis techniques,
- level of inward shear stress generated on surface of soft foundation,
- location of reinforcement layer in relation to location of soft foundation.

3.1.1 *Suitability of different analysis techniques*

Three analysis techniques can be used to analyse basal reinforced embankments on soft foundations – plasticity solutions, limit equilibrium methods and continuum methods.

Plasticity solutions have been developed for two soft foundation cases – shallow soft foundation depth with constant undrained shear strength and deep soft foundation depth with increasing undrained shear strength with depth. Most soft foundation problems can be approximated to either of these two cases. The use of these plasticity solutions, in the form of charts, enables a quick comparison to be made between the unreinforced embankment geometry and the reinforced embankment geometry. A simple calculation can then be made to determine the maximum tension in the geosynthetic reinforcement. Plasticity solutions are good where a first approximation is required, and can easily demonstrate the difference in geometry between an unreinforced and a reinforced embankment.

Limit equilibrium methods have been the most common technique used to analyse basal reinforced embankments. Computer software has made the repetitive calculations associated with this procedure relatively easy and straight-forward. However, it should be noted that limit equilibrium methods suffer from the following limitations;

- The method assumes full bond between the reinforcement and the surrounding soil – this may overestimate the reinforcement tensions in some cases.
- The interpretation of the moment arm used to determine the reinforcement tension can have a significant effect on the calculated magnitude of the reinforcement tensions.
- The method cannot take into account settlements and deformations – these may have a major effect on the performance of the reinforcement and the embankment.
- Care should be taken when applying the method to non-standard geometry problems.

Continuum methods, i.e. finite element and finite difference methods, are the most sophisticated analysis techniques. The major benefit of these methods is that all limit states can be evaluated in one procedure. However, being a sophisticated method the technique requires the input of sophisticated soil parameters, which are generally not readily available. Hence, the method requires some expertise and knowledge of local geotechnical conditions and soil behaviour. A number of different continuum method programs are available but care should be taken to ensure the soil models used match the insitu conditions and that the programs can accommodate large deformations.

3.1.2 *Level of inward shear stress generated on surface of soft foundation*

The presence of the geosynthetic reinforcement at the base of the embankment fulfils two roles. Firstly, the reinforcement absorbs the outward shear stresses of the embankment fill in the vicinity of the embankment side-slopes thus, preventing them from being transferred into the soft foundation. Secondly, the reinforcement imparts an inward shear stress onto the surface of the soft foundation due to its bond and stiffness characteristics. The absorption of both components of shear stress provides the bearing capacity benefit of the reinforced embankment as well as determines the magnitude of the reinforcement tension.

While the absorption of the outward shear stresses due to the embankment fill is well-recognised and defined, the generation of inward shear stresses at the surface of the soft foundation is more complicated because it relies upon the maintenance of an effective bond between the reinforcement and the soft foundation, and upon the differential stiffness characteristics between the soft foundation and the reinforcement. To impart inward shear stresses on the surface of the soft foundation the reinforcement stiffness must be greater than the stiffness of the soft foundation soil – the relative stiffness of the reinforcement restrains the horizontal movement of the soft foundation soil. For high strength geosynthetic reinforcement the stiffness is normally greater than that of most soft foundation soils except for those soils that are highly sensitive or show brittle behaviour. Thus, potentially, high strength geosynthetic reinforcement has the ability to generate these inward shear stresses provided it can maintain an effective bond with the soft foundation soil.

In practice, the bond between geosynthetic reinforcement and the soft foundation soil is not perfect. This has to do with issues of adhesion and strain compatibility. Jewell (1996) recommends practical upper limits of bond coefficient of +0.5. It is to be emphasised that this bond coefficient value is an upper practical limit and that in practice bond coefficients may be lower than +0.5 – especially for more extensible geosynthetic reinforcements.

The use of geosynthetic reinforcement to absorb the outward shear stresses due to the embankment fill only can be considered a "lower-bound" reinforcement solution inasmuch as this is the minimum reinforcement effect that will occur – proper geosynthetic reinforcement will always provide this reinforcing component. Whether the reinforcement also generates inward shear stresses on the soft foundation soil depends on its relative stiffness and bond characteristics. A maximum upper limit for bond coefficient of +0.5 when deriving the inward shear resistance provides an "upper-bound" reinforcement solution inasmuch as this is the maximum reinforcement effect that will occur. In practice, the actual reinforcing effect will lie somewhere between the upper-bound and lower-bound reinforcement solutions.

Design charts based on plasticity solutions enable a reinforcement/soft foundation soil bond coefficient to be chosen, thus enabling a limit to be placed on the magnitude of the resulting reinforcement tension. Limit equilibrium methods based on the slip circle approach always assume that full bond is realised and thus may calculate reinforcement tensions greater than what can be achieved in practice. Continuum methods provide the most accurate approach however, good reinforcement/soil interaction models are required within the software.

3.1.3 *Location of reinforcement layer in relation to location of soft foundation*

Standard embankment geometries are considered to have a trapezoidal embankment cross-section founded directly on the soft foundation soil. In these instances the location of the reinforcing layer lies at the base of the embankment fill immediately above the soft foundation soil. In some instances, however, it might be desirable to provide a basal reinforcement solution where the practical location of the reinforcement lies some distance above the surface of the soft foundation soil. Examples of this could be the need to construct an embankment on a soft foundation that is overlain with an existing fill layer, or where the surface of the soft foundation has dried and a desiccated surface crust of some depth exists.

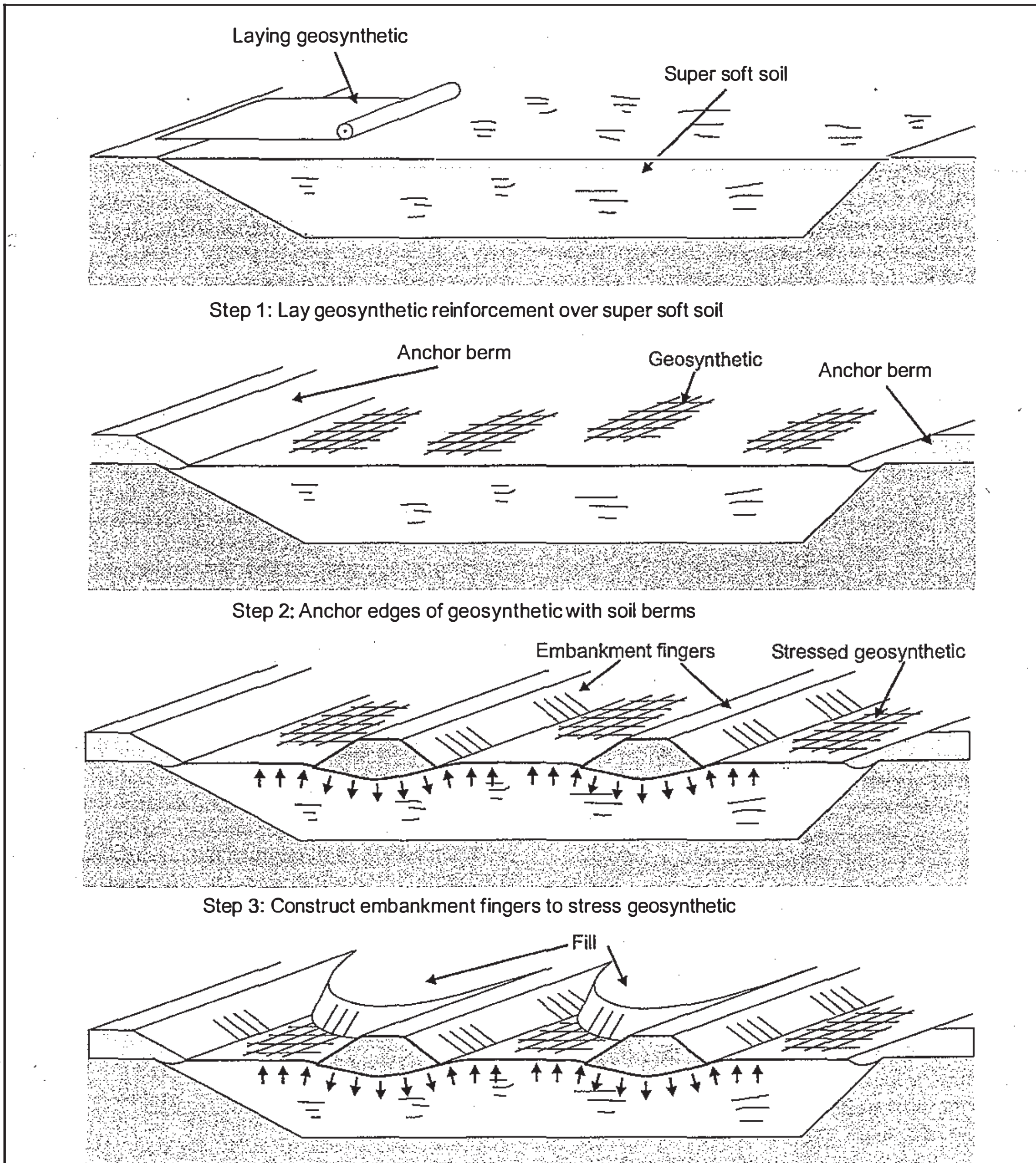

Figure 3. Construction sequence for the reclamation of super-soft soil deposits using geosynthetic reinforcement.

In practice, to provide the most benefit the reinforcement should be installed as close as possible to the surface of the soft soil layer. If this is not feasible then a maximum distance of 1 m is acceptable. Distances greater than this will diminish the reinforcing effect markedly.

3.2 *Geosynthetic reinforced fills over super-soft soils*

The use of geosynthetic reinforcement as a means of capping super-soft soils (so-called sludge capping) first began to be used in ASEAN in the mid 1980's, e.g. Broms & Shirlaw (1987). The use of geosynthetic reinforcement is considered to provide a cost effective means of reclaiming mining waste lagoons (e.g. bauxite and tin mining) in a relatively quick manner. The subsequent fill loading and consolidation of the mining waste lagoons enables the construction of additional structures such as sporting facilities, highway embankments and housing developments.

Super-soft soils exhibit very low undrained shear strengths with insitu moisture contents lying near, or at, the liquid limit. Consequently, the construction sequence must be carried out with considerable care to ensure adequate margins of safety. Four techniques have been used to reclaim super-soft soils. These are;

- Wait until the surface of the super-soft soil has dried out and a crust has formed. This enables construction to be carried out on top of the crust, but can take a considerable period of time waiting for the crust to form.
- Excavate the super-soft soil and replace it with better quality fill. While providing a good foundation, the issue of where to dispose of the super-soft soil (which may contain contaminants) presents a significant problem.
- Stabilise the super-soft soil with cement, etc. The success of deep-mixing the chemical stabiliser with the super-soft soil is dependent on the ability of the chemical stabiliser to react with the super-soft soil thereby increasing shear strength.
- Use geosynthetic reinforcement (with or without prefabricated vertical drains - PVD's) to quickly reclaim the super-soft soil deposit. Because of the very low shear strength of the super-soft soil the construction sequence needs to be site specific. However, the general construction procedure normally follows that shown in Figure 3.

For this application medium to high strength woven polypropylene or woven polyester geotextiles are most commonly used, although other reinforcement types have been used. For example, Toh et al. (1992) report on the successful application of a composite reinforcement layer comprising a nonwoven geotextile in conjunction with bamboo laid in a lattice network to stabilise the reclamation of very soft soils. In this instance the nonwoven geotextile provided the separation function with the bamboo lattice providing the tensile strength and some bending rigidity. This technique has been applied a number of times to the stabilisation of tin mine tailings and very soft foundation soils in Malaysia and Thailand.

There are other instances where geotextile reinforcement has been used to stabilise super-soft soils. For example, Kam & Rankilor (1996) report on the successful use of a woven polyester geotextile to stabilise very soft silt in order to complete a reclamation area filling for the extension to Changi Airport in Singapore. Here, the reliance was on the tensile strength and stiffness of the geotextile reinforcement to not only stabilise the very soft silt but also to resist the extreme installation stresses.

3.3 *Geosynthetic reinforced piled embankments*

While conventional piled embankments (i.e. unreinforced) were first used in highway structures in ASEAN in the early 1970's (Holmberg 1979), geosynthetic reinforced piled embankments were first used in Singapore in the early 1980's to prevent the differential settlement between piled bridge structures and the approach embankments, Broms & Wong (1985). This example, shown in Figure 4, consisted of timber (bakau) piles installed on a 1 m grid with a double layer of woven polypropylene geotextile placed across the base of the abutments prior to placement of the embankment fills. The geotextile had a biaxial tensile strength of 45 kN/m and the two layers were separated by a 300 mm thick sand layer.

Various applications for geosynthetic reinforced piled embankments are shown in Figure 5. In all cases the reinforced piled foundation is used to prevent the detrimental effects of differential settlements. The types of geosynthetic reinforcements currently used in reinforced piled embankments range from high strength woven polypropylene and polyester geotextiles to geogrids to high strength composite geosynthetic reinforcements.

Currently, there are a number of different design and analysis methods promoted for reinforced piled embankments, and herein lies the confusion because these different methods calculate different reinforcement strengths for the same embankment and pile geometries. The design methods promoted range from more formalised methods, such as BS 8006 : 1995, to simplified hybrid 2-dimensional methods, to pure 3-dimensional methods. Reconciling the differences between the various design methods highlights the major issues associated with geosynthetic reinforced piled embankments. These may be explained in terms of the following three subject areas;

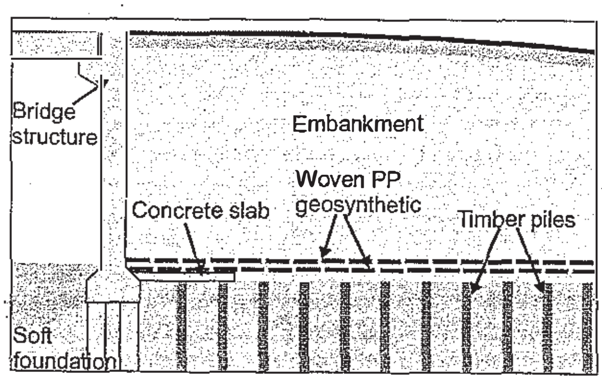

Figure 4. Toa Payoh overpass piled embankment, Singapore, 1984.

- The arching in the embankment and how this is modelled.
- The role of foundation support between the pile caps.
- Serviceability limits associated with low-height piled embankments.

3.3.1 *The modelling of arching in piled embankments*

Piled embankments constitute a complex foundation interaction problem. Along the base of the embankment is the incompressible pile caps interspersed between the soft, compressible foundation soil. This difference in compressibility creates arching in the embankment fill between adjacent pile caps. The accurate assessment of the degree of arching and its effect on reinforcement loads is crucial to the analysis of piled embankments.

Reinforced piled embankments may be analysed as a 2-dimensional (2D) or 3-dimensional (3D) problem depending on the nature of the piled foundation. If the piled foundation consists of a series of pile caps connected by ring beams then the problem should be analysed by 2D means, e.g. Low et al. (1994). If the piled foundation consists of individual pile caps only then the problem should be analysed by 3D means. Analysing individual pile cap layouts by simplified 2D means overestimates the degree of arching and underestimates the reinforcement tensions generated (see Figure 6(iv)) and should be avoided.

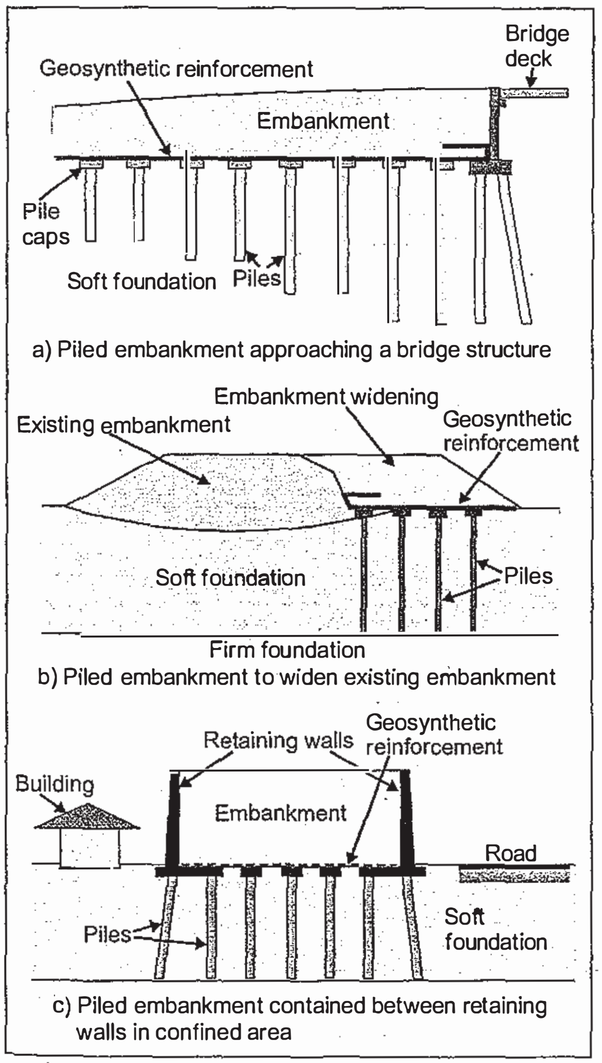

Figure 5. Applications for the use of geosynthetic reinforced piled embankments.

The differences between the 2D and 3D representation of the reinforced piled embankment problem are shown in Figure 6. The 2D representation of the piled embankment problem is a series of beams connecting adjacent piles, Figure 6a(i). This results in a cylindrical arch formation, Figure 6a(ii), with a comparable deformed geosynthetic reinforcement, Figure 6a(iii). The resulting tensions in the reinforcement are shown in Figure 6a(iv). The 3D representation of the piled embankment problem is a series of individual pile caps, Figure 6b(i). This results in a vaulted arch formation, Figure 6b(ii), with a comparable deformed geosynthetic reinforcement, Figure 6b(iii). The resulting tensions in the reinforcement are shown in Figure 6b(iv). For the 2D case the pile cap area ratio in direct contact with the base of the embankment is a/s whereas for the 3D case it is a^2/s^2 which is a much smaller ratio. Consequently, the degree of arching is much higher for the 2D case than for the equivalent 3D case. This results in a much lower reinforcement tension for the 2D case than for the equivalent 3D case (compare Figure 6a(iv) with 6b(iv)).

It should be remembered that the maximum reinforcement tension occurs at the edges of the pile caps and not in the centre of the unsupported spans. Consequently, the calculation of reinforcement tension also needs to accurately address this.

3.3.2 *The role of foundation support between the pile caps*

Current design procedures disregard the presence of the soft foundation between the pile caps when determining the loads in the geosynthetic reinforcement. Foundation support beneath the geosynthetic reinforcement has a marked effect on reducing the loads carried by the reinforcement, Jones et al. (1990). Thus, the current design procedures calculate relatively high tensile loads in the reinforcement because of the assumption of no foundation support. The fundamental question is, can foundation support be relied upon to exist in practice?

As the embankment is constructed on top of the reinforcement the reinforcement must be able to deform to transfer that part of the embankment loading not supported directly by the pile caps. Initially, this loading is transferred directly to the soft foundation between the pile caps which causes settlement of the soft foundation surface between the caps. At some point in

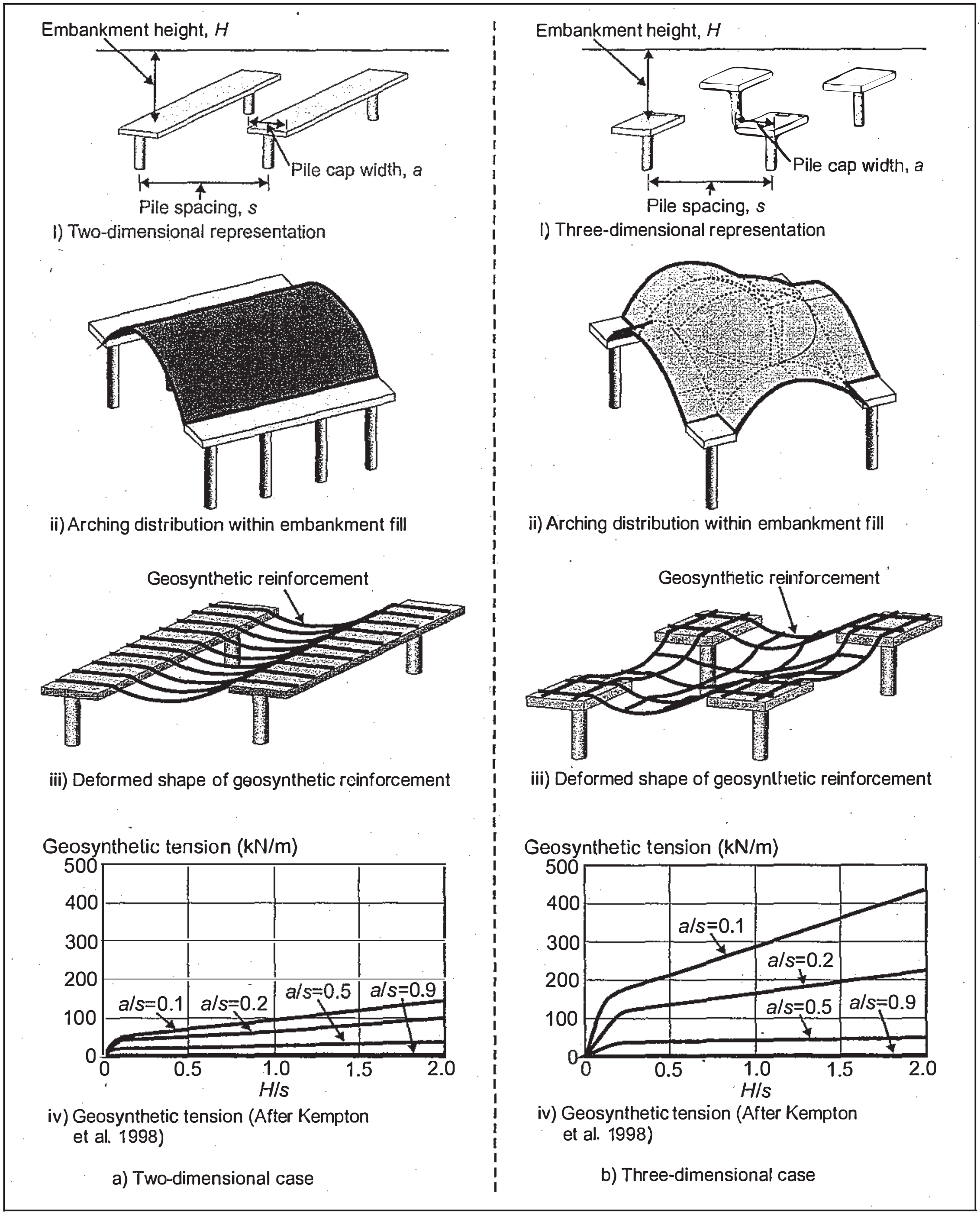

Figure 6. The difference between 2D and 3D representation of reinforced piled embankments.

time an equilibrium condition is reached where the deformed reinforcement supports the unarched embankment loading and negligible embankment loading is applied directly to the soft foundation. Thus, over time it would be safe, and conservative, to assume that the soft foundation soil provides negligible support for the embankment fill.

3.3.3 *Serviceability limits associated with low-height piled embankments*

For low-height piled embankments the most critical limit state is that of serviceability to ensure the surface deformations of the piled embankment remain within acceptable levels, Figure 7. Serviceability can be measured in terms of a maximum allowable surface differential deformation. To meet this serviceability requirement specific limits need to be placed on minimum embankment height, maximum pile cap spacing and minimum reinforcement tensile stiffness and strength.

In general, the serviceability limits of low-height piled embankments have not been addressed well in ASEAN. While BS 8006 : 1995 places embankment height and pile cap spacing limitations for serviceability requirements the approach is simplistic, and consequently, a more rigorous approach is warranted. Research is currently underway to determine acceptable serviceability levels for different combinations of embankment height, pile cap spacing and reinforcement properties. It is anticipated that the results will be comparable to those obtained by Lawson et al. (1996) for reinforced embankments spanning foundation voids, which is a similar problem.

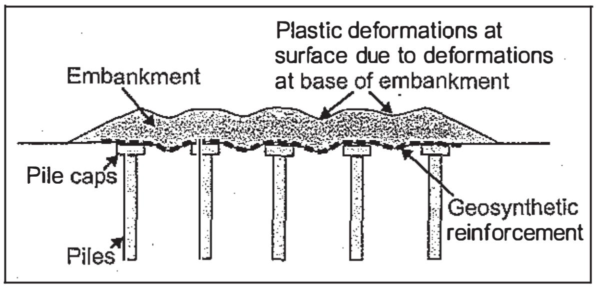

Figure 7. Diagram showing serviceability limit state for low-height piled embankments.

4 REINFORCED FILL SLOPES

Reinforced fill slopes were first used in ASEAN in the mid 1980's in Indonesia and Malaysia. Since that time the technique has been used in all of the ASEAN countries. While the reinforcements used in the earlier slopes were either woven geotextiles or (the then new) geogrids, current reinforced slopes predominantly contain geogrids as the geosynthetic reinforcement. Today, reinforced fill slopes is a simple and straight-forward technique.

An example of the technique is shown in Figure 8 where layers of geogrid are used to construct a stable steep slope. Soil bags were used to provide local stability for the slope face and a geomat was used on the face to promote vegetation growth. An unusual feature of this example is the use of two layers of reinforcing geotextile at the base of the slope to protect against a possible overall stability failure.

The use of reinforced fill slopes has not been without its difficulties in ASEAN. These difficulties have

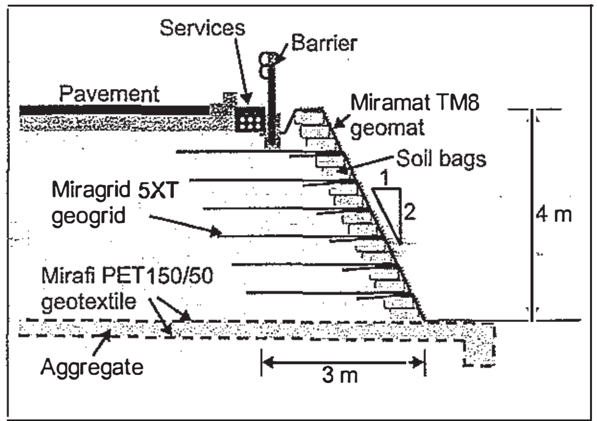

Figure 8. Reinforced slope, Sentosa, Singapore.

arisen due to the following;

- A lack of appreciation of the importance of slope facing,
- Confusion as to which design procedure to use,
- Recognising the difference between standard and non-standard slope geometries,
- Using local fills in a tropical environment,
- Recognising the importance of drainage.

These issues are discussed in further detail below.

4.1 *The importance of slope facings*

Slope facings are an integral component of reinforced slope design and, depending on the nature of the slope, can play an "active", i.e. structural, role or a "passive", i.e. non-structural, role. A classification of reinforced slopes along with the types of facings used is given in Figure 9. Reinforced slopes can be classified as being steep or shallow depending on their slope angle. Steep slopes are where the tension in the reinforcements has not dissipated on reaching the slope face and hence an "active" (structural) facing is required. Shallow slopes are where the tension in the reinforcements has dissipated before reaching the slope face and hence a "passive" (non-structural) facing is only required. The region of $\beta \approx 45°$ provides the transition between steep and shallow slopes. Very steep slopes, i.e. $\beta \geq 70°$, may be designed as retaining walls.

Examples of active facings are wrap-around (with and without soil bags), steel mesh, geocell and gabion facings. Examples of passive facings are vegetation, geomat reinforced vegetation and geoblanket reinforced vegetation. In a number of instances active facings also include passive facing components in order to promote vegetation growth on steep slopes. Because of its tropical climate passive facings should be the minimum face requirements for all reinforced slopes in ASEAN.

It is important that where passive facings consisting of geomats and geoblankets are used intimate contact be maintained between geomat/geoblanket and

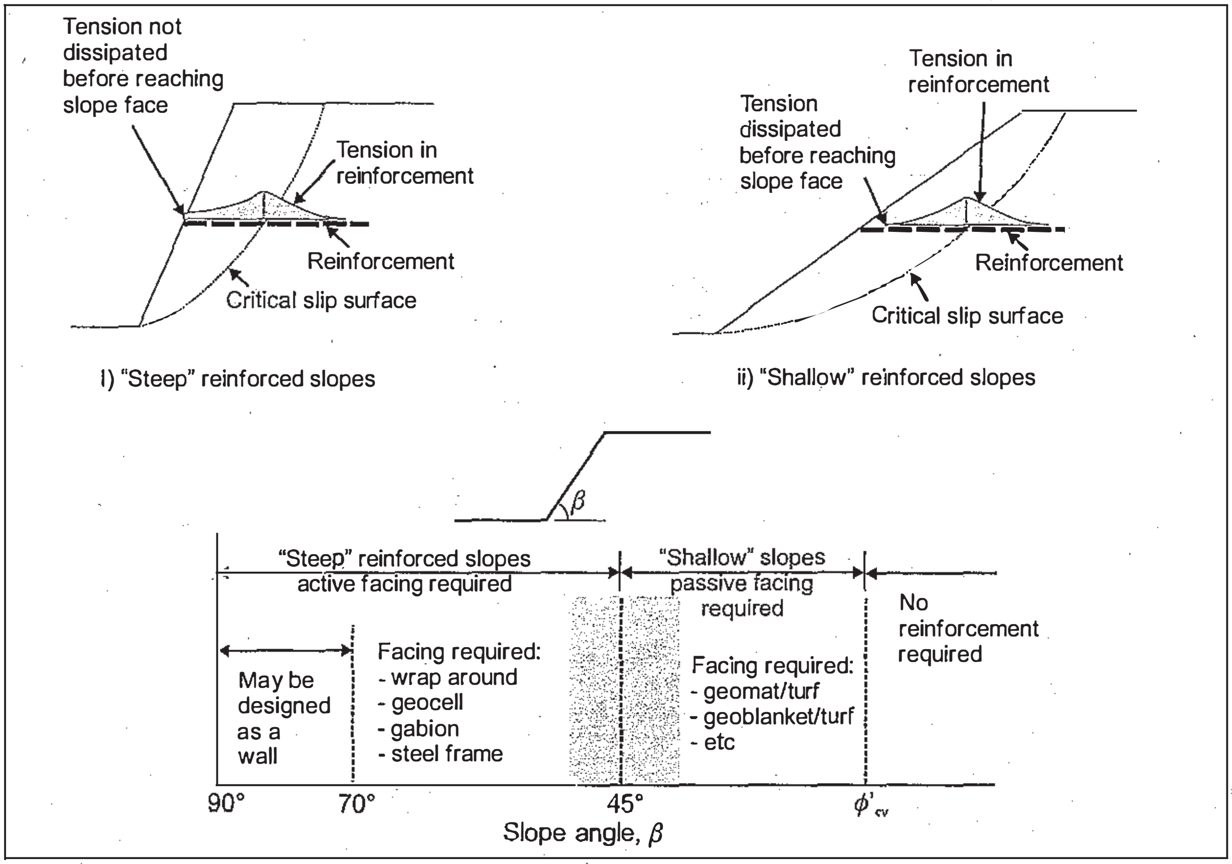

Figure 9. Classification of reinforced fill slopes according to BS 8006 : 1995.

the exposed soil surface. This facilitates vegetation growth and prevents localised erosion beneath the geomat/geoblanket. Good connections at close intervals between the geomat/geoblanket and the soil surface of the slope are crucial. Furthermore, the geomat/geoblanket should also be connected to the extremities of the reinforcement layers at the slope face.

In some parts of ASEAN it has become the practice to over-build the slope face and then regrade the slope to the required slope angle. This is done to ensure good compaction of the fill in the region of the slope face. The reinforcement layers are cut back smooth with the slope face. While a good, stable face is obtained with this approach it is important to remember that this technique is only applicable for shallow slopes (i.e. slope angles ≤ 45°). Furthermore, the compacted soil face should not be left exposed, but covered with a passive facing.

4.2 *Design procedures used*

Today, the design of reinforced fill slopes is a simple and straight-forward procedure based on well-recognised analytical techniques. These techniques have been formalised in a number of design codes, e.g. BS 8006 : 1995. The two most common techniques are the two-part wedge and log-spiral analysis methods and these give fairly consistent results provided appropriate material and load parameters are used and realistic safety factors are included. Today, these analysis methods are available as computer software, which makes the analysis and design of reinforced slopes very easy.

Other design methods are available, for example, Jewell (1990) has presented a series of design charts based on the two-part wedge analysis of standard slope geometries (see Figure 10). These design charts are in common use the world-over and have been used for many reinforced slopes. These design charts have also been converted to spreadsheet format, which makes the optimisation of reinforcement layouts ever easier.

More sophisticated analysis techniques based on continuum methods are also available but require sophisticated input parameters.

4.3 *Standard versus non-standard slope geometries*

In designing reinforced slopes a differentiation needs to be made between what is termed a "standard" slope and what is termed a "non-standard" slope. Standard slopes have relatively simple geometries and loading profiles, Figure 10a. Non-standard slopes can have relatively complex geometries, material characteristics and loading profiles, Figure 10b. Standard slope

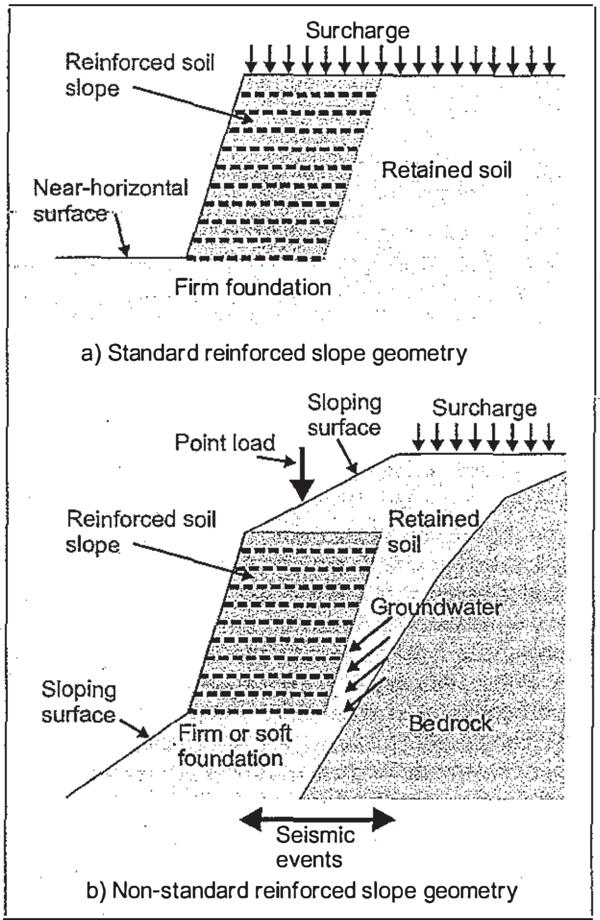

Figure 10. Standard and non-standard reinforced slope geometries.

geometries are by far the most common and many situations can be reduced to this form of geometry. The chart and spreadsheet design methods make these simple geometry slopes very easy to design for, with most of the work taken up in detailing.

Non-standard slope geometries require not only more complex analysis procedures, but also a more detailed understanding of the slope environment. It is incorrect to apply the simple, standard slope design procedures to these more complex slope geometries, Lawson & Yee (1998). Non-standard slope features that require a more rigorous analysis are slope heights over 15 m, sloping soil surface above and/or below the reinforced zone, soft foundation conditions, exterior line or point loads, the occurrence of bedrock near the heel of the reinforced zone, and seismic events.

4.4 *Use of local fills in tropical environment*

Reinforcement can be used to improve the shear resistance of all soil types. However, for poorer quality soils (soils with lower friction angles) the amount of reinforcement that is required to provide a given shear resistance improvement increases considerably. Thus, better quality soils require less reinforcement for stability. (One reason why specific granular fills are specified for reinforced soil retaining walls is because the quantity of reinforcement required to ensure stability is relatively small.)

One of the major economic benefits of geosynthetic reinforcement is that it allows the extensive use of local soils in reinforced soil structures. In the ASEAN region the most common form of local fill is obtained from residual, or saprollytic, soil deposits. These soils generally exhibit good friction angles (33° $\leq \phi' \leq 38°$) and can be highly stable fills provided they are placed with good compaction. In addition, sands obtained from river beds or tin mining operations also make good structural fills, as does quarry dust.

During periods of heavy rainfalls water infiltration and groundwater can have a deleterious effect on placed residual soil fills. For example, many road embankments constructed in hilly terrain in ASEAN suffer from slips during periods of heavy rainfall. The

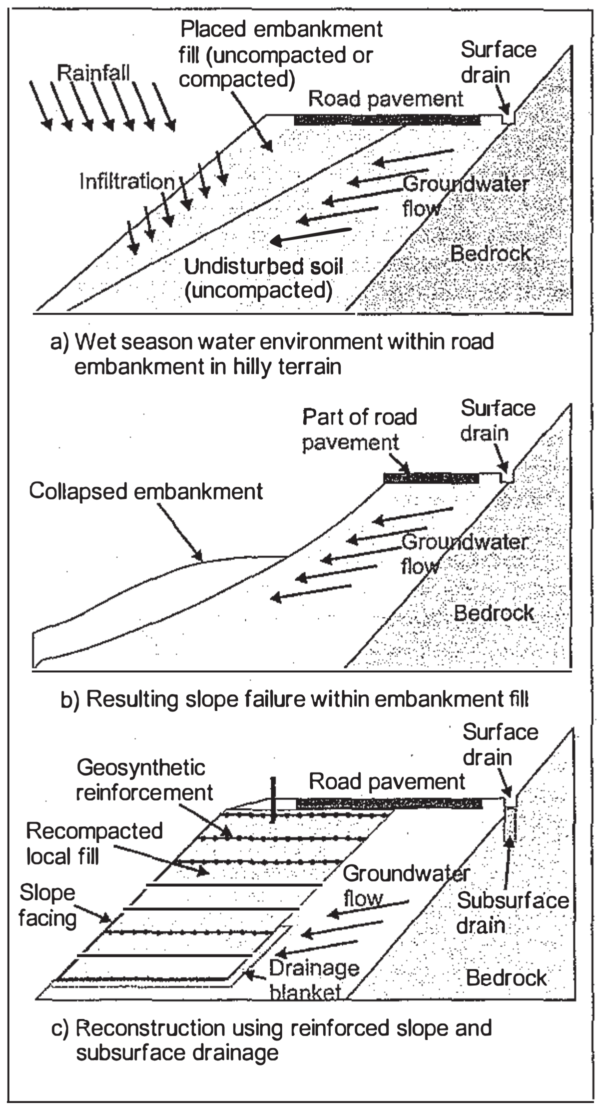

Figure 11. Slope failures and corrective measures in hilly terrain in ASEAN.

problems arise due to a combination of surface infiltration into uncompacted fills and groundwater flows through existing soil deposits within highway embankments, Figure 11a. The build-up of water pressure within the embankment fills leads to collapse, Figure 11b. A combination of reinforced soil slopes and good subsurface drainage provides an economic means of reconstructing these failed slopes. The local fill material can be recompacted, along with the placement of geosynthetic reinforcement, to reconstruct a stable embankment, Figure 11c. Subsurface drainage is required to ensure that groundwater is removed from behind the relatively impermeable reinforced zone, thus preventing a possible build-up of water pressure. This technique has proved very successful in the reconstruction of a number of failed highway slopes in hilly terrain in the ASEAN region.

4.5 *The importance of drainage*

As shown in Figure 11, drainage plays an integral part in good reinforced slope design, especially where local soils are used for reinforced fills. Good drainage (both surface and subsurface), combined with good compaction, enables many of the local soils in ASEAN to be used as structural fills in reinforced slopes.

Where high reinforced slopes are constructed in several tiers surface and subsurface drainage galleries should be included at each tier, with the subsurface drainage galleries extending all the way to the extent of the excavation, Figure 12.

Of recent times interest has been shown in geosynthetic reinforcements that combine the functions of reinforcement with in-plane drainage. It must be remembered that these materials are no substitute for "active" subsurface drainage measures as their hydraulic conductivity is too small to remove groundwater from around reinforced fills. Rather, these materials may remove local build-ups in pore water pressures within the compacted fills only – i.e. they are "passive" drainage materials.

5 REINFORCED SOIL RETAINING WALLS

The first reinforced soil technique to come to ASEAN was the proprietary retaining wall technique of Terre Armee in the early 1980's. This technique utilises concrete segmental panels and steel strip reinforcement. Today, this retaining wall technique is standard practice in many ASEAN countries.

Over the last five years a number of proprietary segmental block retaining wall systems utilising geosynthetic reinforcement have become available in the ASEAN region. Segmental block retaining walls are highly economic structures and are very easy to construct. Typical features of a geosynthetic reinforced segmental block wall are shown in Figure 13. The segmental blocks in the facing are installed by hand and align themselves by the use of pins or a shear key. For walls over 2 m in height geosynthetic reinforcement is used within the compacted backfill to provide stability to the structure. Depending on the type of soil backfill used a drainage layer may be included behind the block wall face. The drainage layer aids friction with the geosynthetic reinforcement and enables local water to drain away from the wall face. The most common form of geosynthetic reinforcement used with segmental block walls is a geogrid.

Geosynthetic reinforced segmental retaining walls have been constructed in Indonesia, Singapore, Malaysia, Brunei, Thailand and the Philippines. One such example is shown in Figure 14 where a 5 m high segmental block wall was constructed to replace a failed masonry retaining wall. It is expected that this tech-

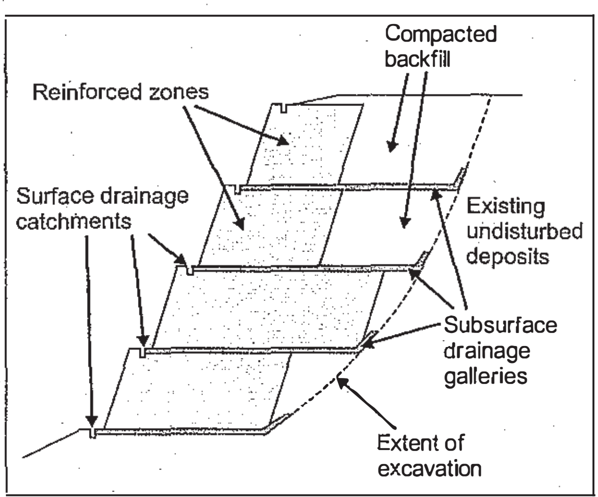

Figure 12. Recommended drainage layout in steep, high, reinforced slopes (After Lawson & Yee 1998).

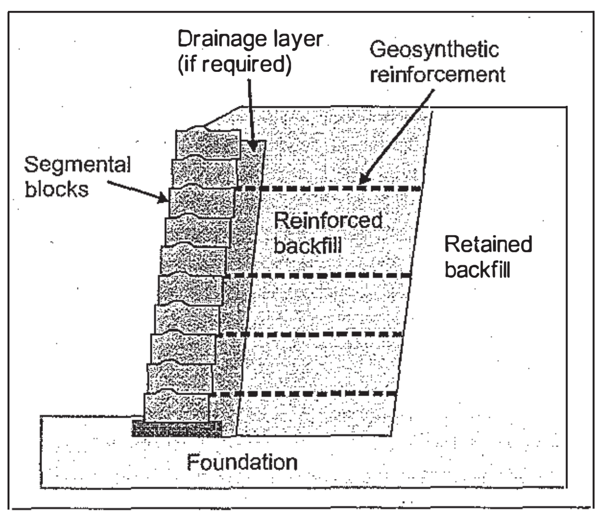

Figure 13. Typical features of a geosynthetic reinforced segmental block retaining wall.

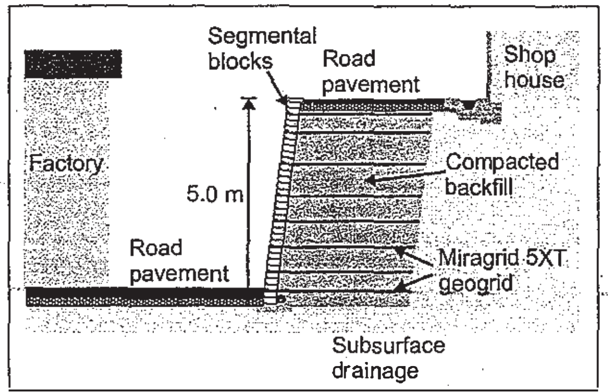

Figure 14. Geosynthetic reinforced segmental retaining wall, Belakong, Malaysia.

nique will become a common form of retaining wall construction over the next few years.

The technology of segmental block retaining walls emanated from North America and a generic design code has been published, NCMA (1995). However, the design procedure in this code produces different reinforcement strengths and quantities to the well-recognised European codes, e.g. BS 8006 : 1995. The reason for this is the different assumptions made regarding the reinforced soil zone behind the wall face. The NCMA code assnmes there is no influence of the retained backfill when calculating the vertical stresses acting on the reinforcement layers. Furthermore, the NCMA code allows for some vertical friction at the back of the facing, and takes into account the natural laid-back angle of the segmental block face when calculating the horizontal thrust of the reinforced fill. The culmination of these differences results in reinforcement savings of approximately 20% over the more conservative European codes. Even with these savings, current research is demonstrating that the NCMA method is still a conservative design method for these types of reinforced soil walls.

A variety of backfills may be used with reinforced segmental retaining walls, however, in ASEAN's wet, tropical climate granular fills of sand or quarry dust provide the easiest option. The reason for this is that good engineering characteristics are obtained with these materials and simple construction processes can be performed when using these fill types. However, this should not rule out the use of other fill types. For example, residual soils have been successfully used as the reinforced fill in geosynthetic reinforced segmental block walls, Anuar & Faisal (1998). Good control of moisture content and good compaction is important for the good performance of these soil types.

Where groundwater is encountered adequate subsurface drainage across the base and behind the reinforced zone should be provided. It is important to remember that the drainage layer commonly located immediately behind the block facing (see Figure 13) should not be relied upon to remove groundwater from the retained backfill – the groundwater should be removed before it enters the reinforced zone.

6 CONCLUSIONS

In the ASEAN region geosynthetic reinforced structures have been used for almost 20 years. While a number of issues still need to be resolved regarding ~~the different reinforced soil applications it~~ is expected that the reinforced soil technique will have a bright future in the ASEAN region.

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