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Performance related issues affecting reinforced soil structures in Asia

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ABSTRACT: In its modem context the reinforced soil technique has been used in Asia for 30 years. While it has been well proven that the technique provides sound, economical, engineering solutions some performance related issues still remain, primarily concerning deformations. To assess the performance of internally reinforced soil structures in terms of deformations, a framework is presented that accounts for the effects of deformation magnitude, time, and serviceability and structural limits. From the perspective of practice, the paper discusses the major performance related issues for four major soil reinforcement applications in Asia – basal reinforced embankments on soft foundations, reinforced piled embankments, reinforced soil retaining walls and reinforced fill slopes.

1 INTRODUCTION

The modem reinforced soil technique first came to be used in Asia in the late 1960's. During the succeeding 30 years the technique has evolved to incorporate new applications and materials.

For the purposes of this paper Asia is considered to cover the four geographical regions shown in Figure 1 – North East Asia, ASEAN (Association of South East Asian Nations), South Asia and Australasia. While this is not a geographically pure representation of Asia (technically Australasia is a separate continent) it provides a convenient means of coverage of the overall region.

The Asian region described in Figure 1 has a wide diversity of features. This region is home to 60% of the earth's population. While most of the countries in the region can be described as having developing economies, some countries, such as Japan, Australia and Singapore have advanced economies. Climate ranges from hot/wet equatorial/tropical to seasonal tropical/temperate to semi-arid to arid conditions. From a geotechnical perspective soils range from very soft marine clays to residual soils to loess to highly overconsolidated clays to granular soils, while rocks range from the most recent to the oldest on the planet. Reinforced soil has come into contact with all of these features at some point over the last 30 years. Their effect on the performance of reinforced soil structures has been varied.

Retaining walls utilizing segmental concrete panels and metallic strip reinforcement were the first modern reinforced soil application used in Asia, beginning in the early 1970's. Reinforced slopes and basal reinforced embankments began to be used during the 1980's, with reinforced segmental block retaining walls starting to be used during the early to mid 1990's.

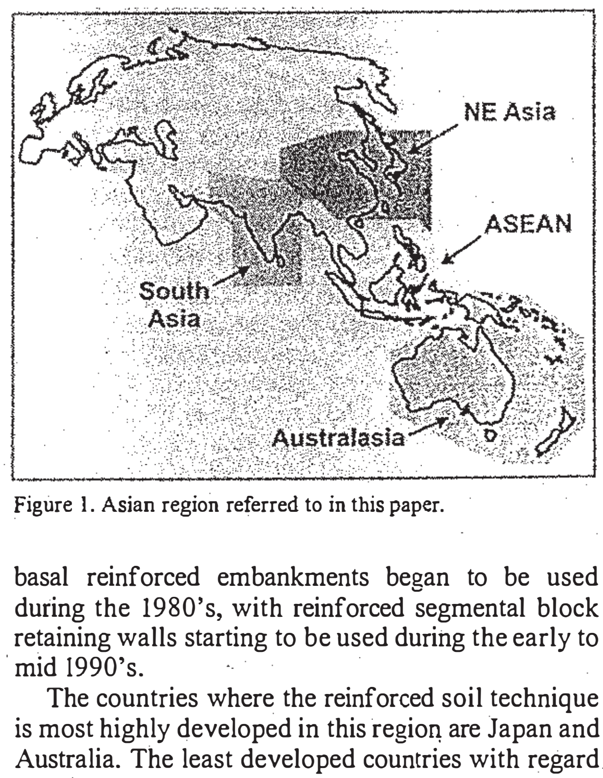

Figure 1. Asian region referred to in this paper.

The countries where the reinforced soil technique is most highly developed in this region are Japan and Australia. The least developed countries with regard to this technique are Myanmar, Kampuchea and Pakistan. In the other Asian countries the reinforced soil technique shows varying degrees of development.

This paper is not meant to be a full treatise on the subject of reinforced soil in Asia. Rather it will highlight specific issues that repeatedly affect the performance of these structures in Asia. While these issues will be presented in an Asian context many are also relevant to other regions. The paper will approach these performance issues from the viewpoint of current practice.

It should be emphasized that the vast majority of reinforced soil structures constructed in Asia have performed very well. The cases referred to in this paper where problems have been observed are a small proportion of the reinforced soil structures constructed. These cases are listed to highlight various performance issues with a view to further improving and refining soil reinforcement practice.

2 REINFORCED SOIL COMPONENTS ACT TOGETHER AS A "SYSTEM"

With reinforced soil structures it is important to remember that the reinforcement, soil and other structural components work together as a composite *system*, and each component contributes to the performance of the overall structure. This is what distinguishes reinforced soil from similar techniques, e.g. soil anchors. From a practical perspective it is worthwhile considering the relative contribution of each of the components to the internal stability of the composite system. This enables a quick judgement to be made of the viability of a reinforced soil solution to a particular problem.

Jewell & Wroth (1987) have shown that all soils may be reinforced, with the poorer quality soils requiring more reinforcement in order to obtain a given stability improvement. Taken to the extreme this could be construed to mean that any soil type could attain any required level of internal stability provided enough reinforcement was included. In practice this is not the case.

While all soils can be reinforced, there are upper practical limits to the amount of reinforcement that can be included to meet a desired level of stability. These limits are governed by performance limitations as well as economics. As a general rule, the soil itself contributes a minimum of 65% of the total shear resistance of the reinforced soil system, with the reinforcement contributing the remainder. For example, if a particular structure is required to have a shear resistance factor of safety of 1.3 and the soil alone results in a factor of safety of 1.0, then reinforcement may be included to provide the additional 30% to arrive at the required factor of safety of 1.3. However, if the soil alone in the same structure only results in a factor of safety of say 0.8, then the addition of reinforcement to provide the required 60% increase in shear resistance is unlikely to provide an acceptable solution. In this case other means are required, in addition to the inclusion of the reinforcement, to provide an acceptable solution, e.g. change in geometry, change in fill type, etc.

Thus, in a reinforced soil system the majority of the internal shear resistance is provided by the soil – the reinforcement is a minor contributor.

3 DESIGN METHODOLOGIES USED IN ASIA

Today in Asia the reinforced soil technique is used in three families of applications – basal reinforced embankments over soft foundation soils with or without piled foundations, reinforced soil retaining walls and abutments, and reinforced fill and soil nailed slopes. These are shown diagrammatically in Figure 2.

Much of the design methodology used in Asia for reinforced soil applications has originated from Europe. More recently, there has been some influence from North America, in particular, with the design methodology for reinforced segmental block walls. In several countries these existing design methodologies have been modified to suit local conditions, practices, requirements, etc.

3.1 *Design/analysis methods used*

The design/analysis methods used in Asia for reinforced soil structures have been based on the well-proven limit equilibrium methods. Essentially, this methodology has been imported from Europe, and later from North America, in the form of either generic, proprietary or hybrid design/analysis procedures. The evolution of these three forms of design/analysis procedures has lead to different design approaches being adopted in different countries in Asia, and for different reinforced soil applications.

The earliest design methods used were either proprietary procedures or simple generic methods for retaining wall design. Later, as reinforced soil design methods developed, more detailed procedures were published covering a wider range of reinforced soil applications. Today, the publicly recognized reinforced soil design procedures are well-developed.

In recent years there has been more use made of continuum-based design/analysis methods due to their improved ease-of-use and their ability to better model complex insitu conditions. In particular, continuum-based methods have almost completely taken over from limit equilibrium methods for the design/analysis of basal reinforced embankments.

3.2 *Design codes used*

Several national and state design codes have been developed in Asia for reinforced soil structures. In all instances, much of the information has been derived from European and, to a lesser extent, North American practice. The well-established codes have also made modifications to suit local conditions and requirements. As with other regions, the most codified reinforced soil structures are retaining walls, while the

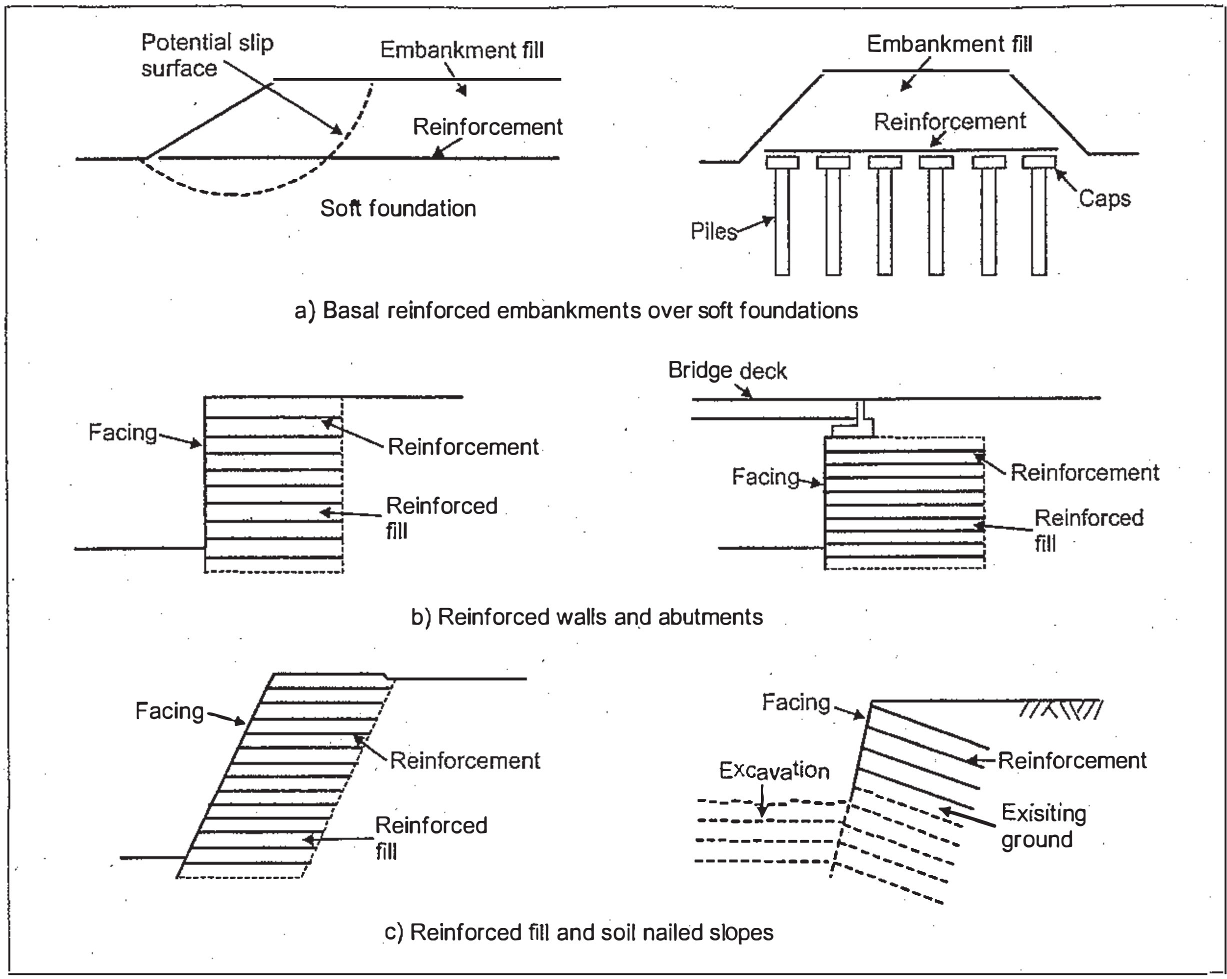

Figure 2. Typical uses of the reinforced soil technique in Asia.

least codified are basal reinforced embankments on soft foundations.

The places where regulated procedures are widespread normally have the most developed design codes. For instance, the New South Wales Department of Transport, Australia, has developed a design code for reinforced soil retaining walls, RTA (1997). This code, based on limit equilibrium methods, is becoming the standard design code for all government projects in Australia and New Zealand. In Hong Kong two design codes exist; one for reinforced soil retaining walls, GCO (1989), and one for reinforced fill slopes, GEO (1993). In Japan a number of Construction Ministry design codes exist; several exist for reinforced soil retaining walls using proprietary facing and reinforcement elements, e.g. PWRC (1990), and one for geosynthetic reinforced soil structures in general, PWRC (2000). In the English speaking countries of ASEAN and South Asia, e.g. Singapore, Malaysia, Brunei, India, Bangladesh and Sri Lanka, the code BS 8006 : 1995 is used. This code is also used for reinforced slopes and basal reinforced embankments in Australia and New Zealand. Other countries, such as Korea and China, are also working on the development of standard design procedures.

In other Asian countries with unregulated design procedures differing design procedures are used depending on the experience of the designer. These design procedures may range from hybrid methods proposed by a material supplier to recognized procedures emanating from the country of origin of the designer or the designer's company.

3.3 *Global factor of safety versus partial factor design methods*

Design methodologies based on working stress methods utilize a global factor of safety approach, whereas those based on limit state utilize a partial factor approach. The countries and states currently adopting limit state design methods are those where European influence is strong, e.g. Australasia, South Asia, ASEAN, and Hong Kong, whereas those countries adopting global factor of safety design methods are where North American influence is strong, e.g. Japan, Korea, Taiwan and the Philippines.

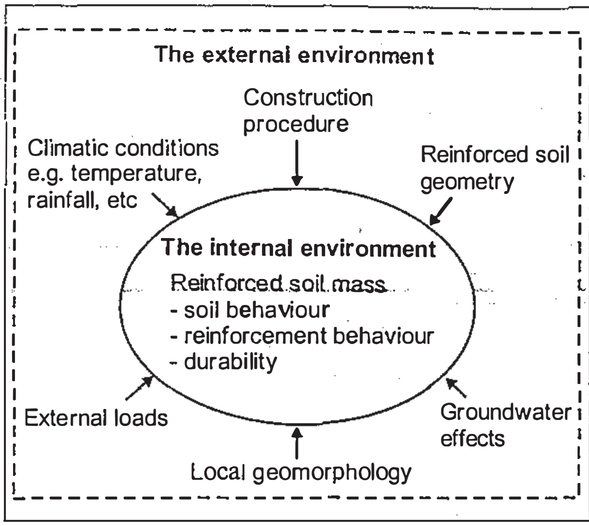

Figure 3. Variables that affect the level of deformation of reinforced soil structures.

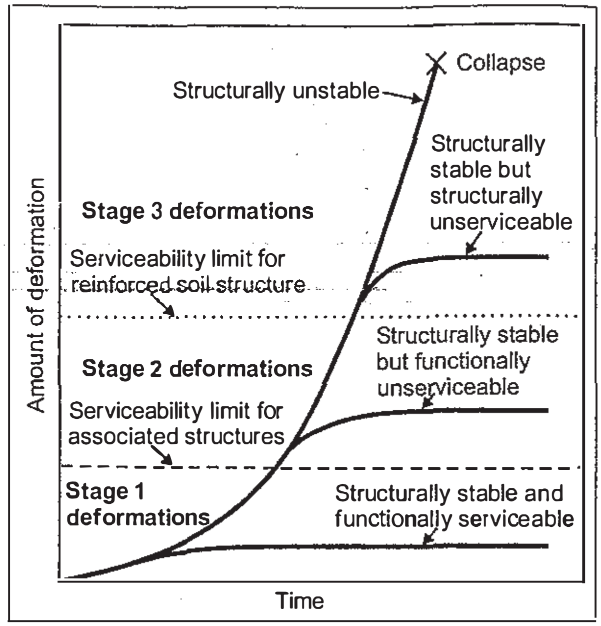

Figure 4. Deformation framework for the performance of reinforced soil structures.

4 DEFORMATIONS ASSOCIATED WITH REINFORCED SOIL STRUCTURES

A major attribute of reinforced soil structures is their flexibility. This flexibility enables them to undergo, and tolerate, significant deformations without impacting their structural integrity. However, this inherent flexibility may impact the performance of other associated structures that cannot tolerate these levels of deformation.

The level of deformation of a reinforced soil structure is governed by its environment. This environment can be divided into two parts; the reinforced soil mass itself – the internal environment – and the environment in which the mass is placed – the external environment, Figure 3. Fundamental to reinforced soil behaviour, the reinforced mass itself must deform in order to redistribute stresses from the soil mass to the reinforcement and back into the soil mass. The amount of deformation occurring in this internal environment depends on the behaviour of the soil and the reinforcement. The maintenance of this level of deformation with time is dependent on the durability of not only the reinforcement but also the soil.

Deformations also arise due to the impact of the external environment on the reinforced soil mass. External variables that impact deformations are external loads, climatic conditions, construction procedure, reinforced soil geometry, groundwater effects and local geomorphology. In many instances the impact of these external variables are underestimated and consequently deformations end up being greater than originally envisaged.

In the field, the performance of reinforced soil structures is assessed by the level of observed deformation the structure undergoes. This is normally done over a specific time period. These deformations are due to a combination of the internal and external environments. To critically assess the performance of reinforced soil structures in relation to the amount of deformation occurring, a framework must be developed that takes into account deformation levels and time as well as the required serviceability limits. These serviceability limits should be established on an objective basis. The format of such a framework is shown in Figure 4.

Figure 4 shows the performance of reinforced soil structures in terms of the amount of deformation and time. The maximum value on the time scale is the design life of the structure. The deformation axis is divided into three stages, each denoting an increasing amount of deformation (and a decreasing performance). The two boundaries that divide the three deformation stages are the serviceability limit for associated structures, e.g. services, pavements, etc., and the serviceability limit for the reinforced soil structure itself. In general, reinforced soil structures can undergo greater deformations than associated structures and still maintain their structural integrity. If a reinforced soil structure continues to deform, its measured performance will pass through each of the three deformation stages.

For reinforced soil structures that undergo only Stage 1 deformations in Figure 4 equilibrium is attained within the serviceability limits of any associated structures. In this case the structure is structurally stable and functionally serviceable. This is the region of ideal performance from the viewpoint of serviceability; however, to achieve this relatively high degree of serviceability a more conservative reinforced soil design approach is normally required.

For reinforced soil structures that undergo Stage 2 deformations in Figure 4 the serviceability limit for associated structures is surpassed but the serviceability limit for the reinforced soil structure itself is still maintained. If associated structures are present then the reinforced soil structure is structurally stable but functionally unserviceable. However, in situations where there are no associated structures in conjunction with the reinforced soil structure then this level of deformation is normally acceptable from a serviceability perspective. If corrective measures are required for Stage 2 deformations then they normally involve the prevention of further deformation of the reinforced soil structure, with the associated structures that have suffered distress undergoing rehabilitation. Techniques involve soil nailing, anchoring, buttressing, etc. In the extreme, dismantling and replacement may be required for Stage 2 deformations, but this is very rare.

For reinforced soil structures that undergo Stage 3 deformations in Figure 4 the serviceability limit for the reinforced soil structure itself is surpassed. The deformations will either continue to a state of structural collapse (the ultimate limit state which may be considered an extreme form of deformation) or may reach equilibrium in which case the structure may be structurally stable but is structurally unserviceable. At these levels of deformation the reinforced soil structure is normally in a state of severe distress. Corrective measures for reinforced soil structures that undergo Stage 3 deformations normally involve dismantling and replacement.

5 PERFORMANCE RELATED ISSUES AFFECTING BASAL REINFORCED EMBANKMENTS ON SOFT FOUNDATIONS

The earliest Asian use of the basal reinforced embankment technique was in Japan during the 1960's (Fukuoka, 1988). Various reinforcement materials were used, ranging from steel mesh to woven geotextiles to geonets, to stabilize embankment fills constructed over soft foundation soils.

This technique was first used in South East Asia during the early 1980's. One of the earliest uses was in the construction of a causeway in Hong Kong in 1982, Figure 5. The causeway, which comprised part of the border fence with the Peoples Republic of China, was constructed over very soft marine clay (undrained shear strength, $s_u = 5$ to 10 kPa) in a tidal environment. To achieve stability of the 3 m high causeway (base width approximately 35 m) basal reinforcement was used in the form of a woven geotextile with an ultimate tensile strength of 200 kN/m. Two types of woven geotextile were used – 90% of the 3.5 km length used a woven polyester geotextile while the remaining 10% used a woven polypropylene geotextile.

This case study has proved interesting because in 1996 two sections of the causeway were excavated to

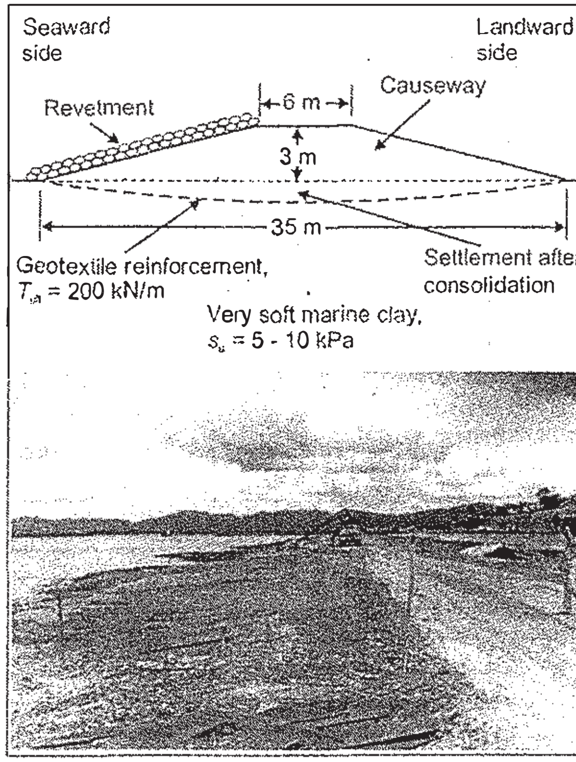

Figure 5. Basal reinforced causeway, Deep Bay, Hong Kong, 1982.

recover the basal reinforcement for analysis of long-term durability, Cowland et al. (1998). The results obtained showed that there had been no strength loss in the woven polypropylene geotextile and only a 15% strength loss in the woven polyester geotextile, even though the geotextiles were not required to maintain their strength long term.

Soft foundations can suffer from inadequate shear strength and high compressibility. The problem of inadequate shear strength arises if the speed of embankment construction is too fast or unbalanced. Solutions to problems of inadequate foundation shear strength involve controlling the geometry and speed of construction of the embankment. The problem of compressibility is not just the amount of settlement but also how long it takes to occur. Solutions to problems of compressibility involve either preventing settlements from occurring or making sure they occur in a short, well-defined, time frame.

Figure 6 lists the various embankment construction methods used where it is necessary to control stability due to inadequate shear strength of the soft foundation. Methods involve stage construction (Figure 6a), berm construction (Figure 6b) and basal reinforcement (Figure 6c). The use of stage construction without any foundation consolidation acceleration treatment can take a long time and consequently this method has lost its attractiveness in recent times, especially with the adoption of new forms of construction contracts that emphasize speed of construction.

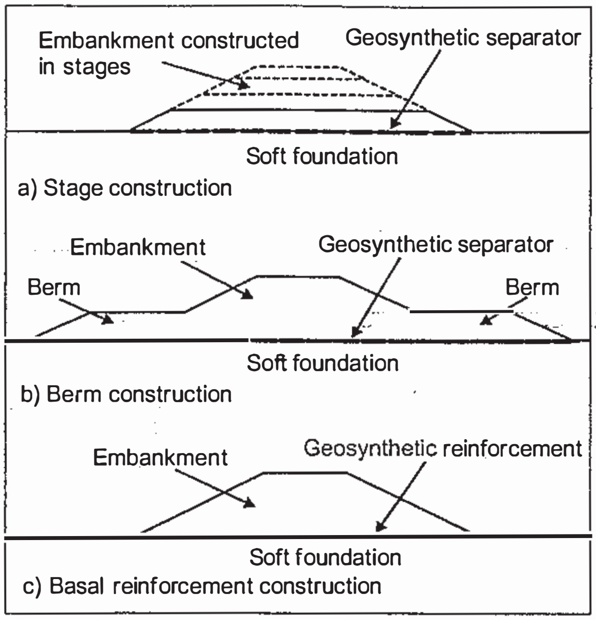

Figure 6. Embankment construction methods to control stability.

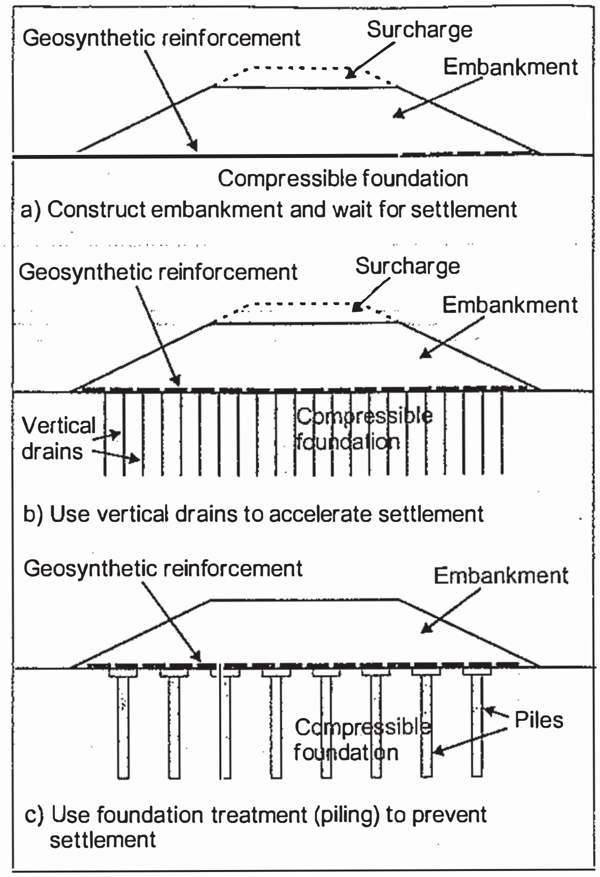

Figure 7. Embankment construction methods to control compressibility.

The use of berms can take up a considerable area. The use of basal reinforcement to control stability has the benefit in that it enables maximization of the embankment height and minimization of the area covered.

Figure 7 lists the various embankment construction methods used to control foundation compressibility. These involve constructing the embankment and waiting for settlements to occur (Figure 7a), accelerating consolidation using vertical drains (Figure 7b) and preventing settlements by adopting a suitable foundation treatment, e.g. piling (Figure 7c). Other methods, such as foundation replacement, also exist. Constructing the embankment and waiting for settlements to occur can take a considerable period of time, even with embankment surcharging. The use of vertical drains to accelerate consolidation has become a common technique as the consolidation period can be shortened to less than a year. This method has the advantage that consolidation settlements can be completed within a relatively short period of time and within the planned construction schedule. If it is necessary to prevent settlements from occurring then a foundation treatment such as piling can be used. This technique is discussed in more detail in Section 6.

It should be noted that basal reinforcement by itself does not control foundation compressibility but can be used in combination with other compressibility treatments to achieve a desired solution in a specific time frame.

The mechanics of basal reinforced embankments on soft foundations is one of bearing capacity. The reinforcement acts in combination with the soils, both soft foundation and embankment fill, to provide an adequate margin of stability. For basal reinforced embankments the maximum practical contribution of the reinforcement to stability is around 35%, with the soil contributing the remainder. More often than not, the reinforcement contribution is around 20-25%. Furthermore, the reinforcement is only required for a relatively short period of time until the foundation soil has consolidated and can support the embankment loading fully. This is quite different to other reinforced soil applications where the reinforcement tension is carried for the full design life of the structure.

New forms of construction contracts, e.g. design, build and maintain (DBM), and design, build, operate and finance (DBOF), are emphasising the merits of basal reinforced embankments and refining their areas of use. Both of these forms of contracts recognise the high cost of future maintenance and the need to minimize it, as well as emphasising speed of construction and quality. With long term maintenance costs to be borne by the designer/contractor/ financier there is a distinct incentive to ensure that all problems concerning stability and, more importantly, settlement are provided for within the relatively short construction period. This makes the use of methods to control foundation compressibility and stability of considerable interest.

Coverage of all of the performance related issues affecting basal reinforced embankments on soft foundation soils would naturally lead to an extensive cov-

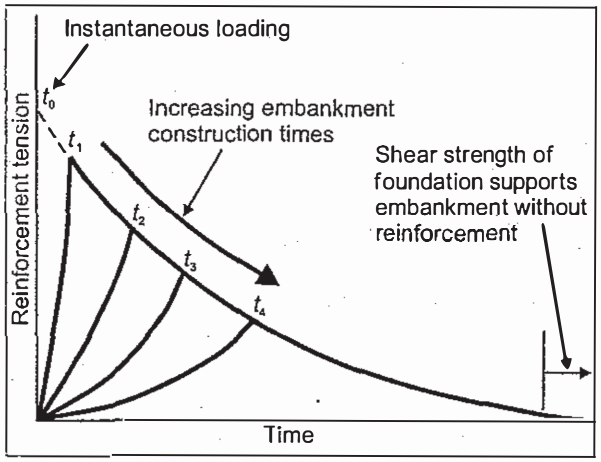

Figure 8. Effect of embankment loading rate on maximum required reinforcement tension.

erage of soil mechanics and foundation engineering principles which is outside the scope of this paper. Instead, four performance related issues will be discussed as these have particular relevance to current Asian practice. These are:

1 The effect of embankment loading rate and foundation consolidation rate on reinforcement tension;
2 The influence of the foundation surface crust on reinforcement tension;
3 The variation and extent of the soft foundation layer;
4 Reinforced fill construction over very soft soils.

5.1 *Effect of loading rate and consolidation rate on reinforcement tension*

Successful embankment construction on soft foundations entails balancing the loading rate of the embankment fill with the consolidation rate, and gain in shear strength, of the soft foundation. The inclusion of basal reinforcement enables a greater rate of embankment loading than would be possible without reinforcement. Refining the basal reinforced embankment design requires linking the loading and consolidation rates along with the required amount of reinforcement.

Figure 8 shows the typical effect of rate of embankment construction on the required reinforcement tension, in particular the maximum reinforcement tension, versus time for constant rate of consolidation. If it is assumed that the maximum tension in the reinforcement occurs at the end of embankment filling, or major stages of loading, then adjusting the embankment construction time can have a marked effect on the maximum reinforcement tension with longer construction times requiring less reinforcement. The reason for this is that the soft foundation is consolidating during embankment loading – the longer the loading period the more consolidation. An "instantaneous"

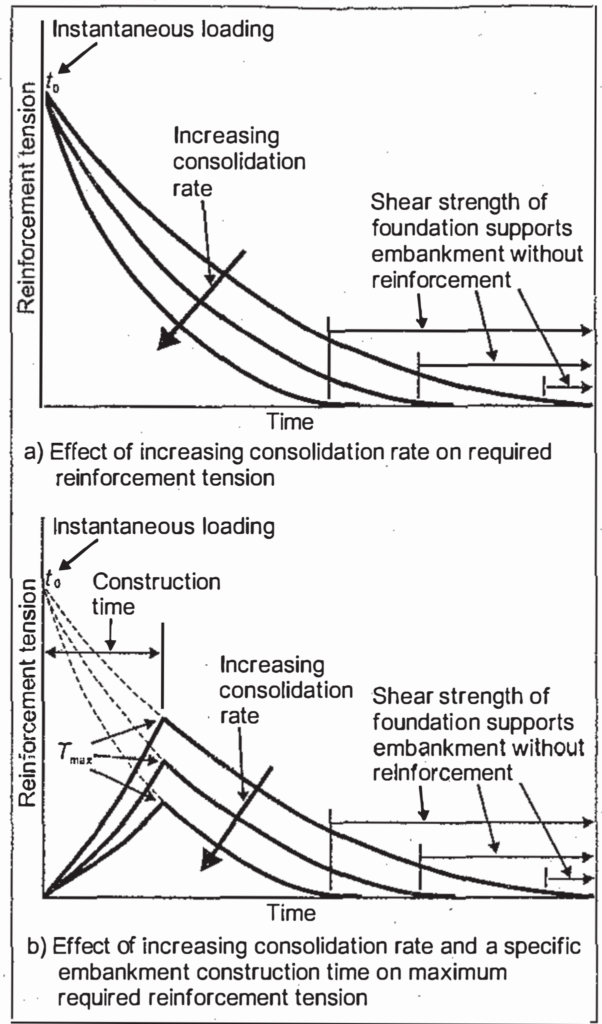

Figure 9. Effect of foundation consolidation rate on maximum required reinforcement tension.

construction time (time t_0 in Figure 8), as is commonly assumed when using limit equilibrium analyses, results in the largest amount of reinforcement. Even a relatively short embankment construction period of one to two months, which is common in practice, can significantly reduce the maximum reinforcement tension required especially if a consolidation acceleration treatment is also applied to the soft foundation, e.g. vertical drains.

Figure 9 shows the effect of increasing the rate of consolidation of the soft foundation on the required reinforcement tension. Increasing the rate of consolidation results in shorter consolidation times and hence shorter periods of time over which the reinforcement is required, Figure 9a. If the embankment loading is instantaneous (time t_0 in Figures 9a, b) then increasing the consolidation rate will not affect the magnitude of the maximum required reinforcement tension, but will affect the period of time over which the tension is required. However, if the embankment loading occurs over a specific construction time then increasing the consolidation rate will affect *both* the magnitude of the maximum required reinforcement tension (T_{max} in Figure 9b) as well as the period of time over which the tension is required. The reason for this is that the foundation is consolidating at a specific consolidation rate during the embankment construction period. When the rate of consolidation is accelerated a relatively short embankment construction period, e.g. one month, can have a significant effect on the maximum required tension in the reinforcement.

The combination of a "managed" embankment construction time along with an acceleration of foundation consolidation provides the ideal reinforced embankment solution where it is necessary to control foundation compressibility within a required period of time as well as stability. As discussed above, and concluded by Yeo & Cowland (1998), there is considerable scope to refine these solutions. Achieving this in the field requires good attention to detail especially if the performance of the consolidation acceleration treatment is critical. A good knowledge of the insitu foundation consolidation parameters is crucial. Where vertical drains are used to accelerate foundation consolidation good attention to drain performance and drainage detailing are also important. Controlling the rate of loading is normally achieved by good site instrumentation and site supervision.

Accounting for embankment loading rate and foundation consolidation rate at the design stage requires the use of relatively sophisticated design/analysis techniques. Realistically, this can only be undertaken using continuum methods. With the improved ease-of-use of continuum method computer programs this technique is becoming more common and cost effective. Today, almost all basal reinforced embankment designs of note are carried out using continuum methods. The economies associated with the appraisal of embankment loading rates and foundation consolidation rates make the time required in arriving at a continuum methods solution worthwhile. However, a word of caution concerning continuum method techniques is that they require sophisticated parameter inputs in order to produce realistic design information. In many instances expert judgement is required for these input parameters.

5.2 *Influence of foundation crust on reinforcement tension*

Except for the recent very soft marine clay deposits, the majority of soft foundation soils in Asia are overconsolidated to some degree. The amount of overconsolidation at the surface of these soft foundation soils can vary from light and ill-formed, e.g. recently deposited silts and clays in a tidal environment, to heavy and well-formed, e.g. a distinct, heavy, vegetated crust.

In simple terms, the overconsolidated crust can be thought of providing some resistance to the disturbing forces of the embankment fill. Thus, where basal reinforcement is used, the crust acts as an additional stabilising measure. In terms of reinforcement resistance, the crust may approximate the following resistance;

$$[T_r]_{crust} \approx 2\sqrt{2}h_c(s_{uc} - s_{us}) \quad (1)$$

where, T_r = reinforcement tension; h_c = thickness of the crust; s_{uc} = undrained shear strength of the crust; and s_{us} = undrained shear strength of foundation beneath the crust. For example, if a foundation crust has an undrained shear strength of 40 kPa, is 1 m thick, and the undrained shear strength of the soft foundation beneath is 20 kPa, then the crust would be equivalent to $2\sqrt{2} \times 1(40-20) = 56$ kN/m reinforcement resistance.

If the crust has a relatively high undrained shear strength compared to the soft soil beneath then its thickness also influences the effectiveness of the basal reinforcement. Ideally, for maximum benefit, the reinforcement should be placed where the maximum horizontal strains occur during loading. These occur immediately beneath the crust at the top of the soft soil layer. Thus, ideally the reinforcement should be placed here for maximum benefit. This, of course is impractical, and the reinforcement is placed on top of the crust some distance from the ideal location, which reduces the effectiveness of the reinforcement and the tension it can carry. For best benefit the basal reinforcement layer should be located within 1 m of the top of the soft soil layer. If it is located over 2 m from the soft layer then the basal reinforcement is of little use. This also has implications where it is being considered to place the reinforcement within the embankment fill and not at the base of the embankment. The same rules apply. The reinforcement should be placed as near to the bottom of the embankment as possible for maximum benefit.

During embankment construction it is very important to ensure that the surface crust is not disturbed. This enables the embankment to be constructed easier and quicker.

5.3 *The thickness of the soft foundation layer*

A fundamental parameter affecting embankment stability and settlement is the thickness of the soft foundation layer. For the same shear strength and compressibility values, the greater the thickness of the soft foundation, the lower the embankment stability and greater the settlement.

Commonly, in the design and analysis of embankments on soft foundations the thickness of the soft foundation layer is assumed to be constant. In practice this is very rarely the case. Considerable variations in soft layer thickness may occur along the embankment length as well as across the embankment width. This variation in soft layer thickness affects embankment stability and settlement, in particular, differential settlement. Site investigations normally follow the centre line of the proposed embankment alignment and, depending on the frequency of the investigation, variations in soft layer thickness may not be detected.

When the embankment extends over a considerable distance a judgement is required on how many sections need to be designed/analysed. In some cases this judgement does not cover all eventualities.

5.4 *Geosynthetic reinforced fills over very soft soils*

In Asia, the use of geosynthetic reinforcement as a means of capping very soft soil deposits (so-called sludge capping) first began to be used in Japan in the 1970's, e.g. Nishibayashi (1982), Yano et al. (1982), and in Southeast Asia in the mid 1980's, e.g. Broms & Shirlaw (1987). The use of geosynthetic reinforcement provides a cost effective means of reclaiming very soft soil deposits, e.g. mining tailings and dredged spoil dumps, in a relatively quick manner. The subsequent fill loading and consolidation of the very soft foundation enables the construction of additional structures such as highway embankments and other developments.

Very soft soils exhibit low undrained shear strengths with insitu moisture contents lying near, or above, 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 very soft soils, namely:

1 Wait until the surface of the very 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.
2 Excavate the very soft soil and replace it with better quality fill. While providing a good foundation, the issue of where to dispose of the very soft soil (which may contain contaminants) presents a significant problem.
3 Stabilize the very soft soil with cement, etc. The success of deep-mixing the chemical stabilizer with the very soft soil is dependent on the ability of the chemical stabilizer to react with the soil thereby increasing shear strength. This technique is also very expensive.
4 Use geosynthetic reinforcement (with or without prefabricated vertical drains - PVD's) to quickly reclaim the very soft soil deposit. Because of the low shear strength of the very soft soil the construction sequence needs to be site specific. However, the general construction procedure normally follows that shown in Figure 10.

For this application medium to high strength woven polypropylene or woven polyester geotextiles are most commonly used, although other reinforcement

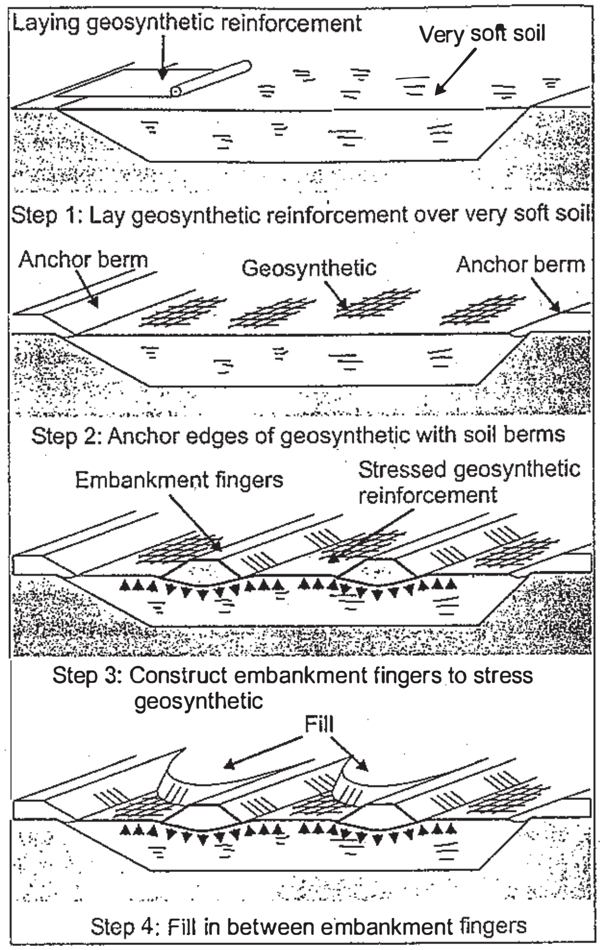

Figure 10. Construction sequence for the reclamation of very soft soil deposits using geosynthetic reinforcement.

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 stabilize the reclamation of very soft soils. In this instance the nonwoven geotextile provided the separation function with the bamboo lattice providing the tensile strength/stiffness and some bending rigidity. This technique has been applied a number of times to the stabilization of tin mine tailings and very soft foundation soils in Malaysia and Thailand.

There are other instances where geotextile reinforcement has been used to stabilize very soft soils. For example, Kam & Rankilor (1996) report on the successful use of a woven polyester geotextile to stabilize very soft silt in order to complete a reclamation 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 stabilize the very soft silt but also to resist the extreme installation stresses.

6 PERFORMANCE RELATED ISSUES AFFECTING REINFORCED PILED EMBANKMENTS

While unreinforced embankments supported on piles were first used in highway structures in South East Asia in the early 1970's (Holmberg, 1978), geosynthetic reinforced embankments supported on piles were first used to prevent the differential settlement between piled bridge structures and approach embankments in Singapore in 1982 (Tan et al., 1985). The reinforced piled embankment, Figure 11, was part of the overpass of Jalan Toa Payoh at the Central Expressway intersection. The foundation profile consisted of soft clay and to minimize differential settlements the area beneath the earth abutments were piled using timber piles. The timber piles were installed on a square grid with spacings varying from 0.7 m to 1.5 m depending on embankment height. Across the top of the timber piles two layers of woven polypropylene geotextile was installed, separated by a 300 mm thick sand layer. The woven geotextile had an ultimate tensile strength of 45 kN/m in both length and cross directions. The geotextile reinforcement and the sand layer were considered to act as a stiff mat to redistribute the weight of the embankment. Subsequent monitoring showed only minimal settlements whereas settlements of the order of 1.0 m to 1.5 m were expected without any foundation treatment.

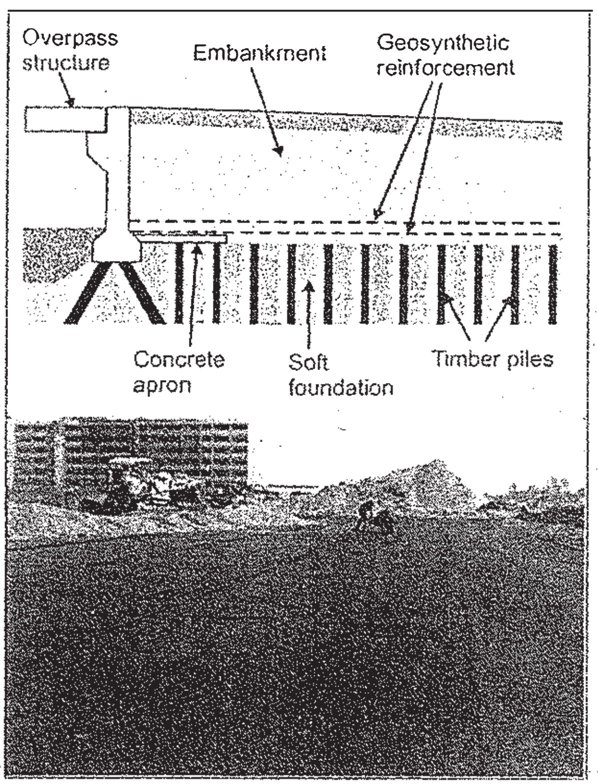

Figure 11. Geotextile reinforced piled embankment, Singapore, 1982.

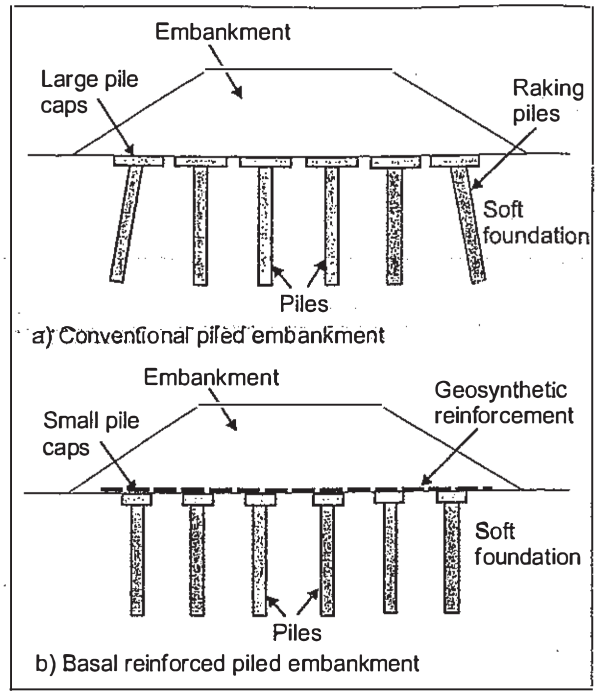

Figure 12. Comparison between a conventional piled embankment and a reinforced piled embankment.

Conventional piled embankments utilize large-size pile caps in order to ensure the entire embankment loading is transferred onto the pile caps by arching, Figure 12a. Furthermore, raking piles are commonly included to counteract the horizontal outward thrust of the embankment fill. This can result in an inefficient design because the pile spacings may not be able to be maximized (to save on cost) due to the required size of the pile caps.

Basal reinforced embankments supported on piles require only small-size pile caps because the reinforcement transfers the loading of the unarched portion of the embankment fill onto the pile caps, Figure 12b. Moreover, the reinforcement also replaces the requirement for raking piles to counteract the horizontal outward thrust of the embankment fill. This results in an efficient design because the pile spacings can be maximized for best economy.

A variety of pile types are used in piled embankments in Asia. The most common are timber piles, and concrete piles are also used, as are stone columns, grout-injected stone columns, concrete columns and jet-grouted columns.

The most common geosynthetic reinforcement used in reinforced piled embankments in Asia is woven polyester geotextiles, but woven polypropylene geotextiles and geogrids are also used depending on the tensile load requirements.

An important feature of reinforced piled embankments is that the reinforcement load is transmitted continuously across the pile caps. To do this the reinforcement should be continuous with preferably no joins. However, if joins are inevitable then an overlap length of a minimum three times the pile spacing should be used.

In Asia, the major performance related issues affecting reinforced piled embankments are concerned with ensuring there is adequate pile group capacity to support the embankment loads; the effect of pile group layout on embankment arching and the consequent determination of reinforcement loads. The role of foundation support between the pile caps and serviceability limits for low-height reinforced piled embankments are also important issues.

6.1 *Pile group capacities and layouts in reinforced piled embankments*

The pile group must have adequate capacity to support the full loading of the embankment. This includes vertical as well as horizontal load capacity. Where the foundation soil is very soft care must be taken to ensure the piles are installed well into the firm foundation stratum beneath. Furthermore, the extent of the pile group must ensure that no instability occurs within the operating area of the surface of the embankment. This normally requires that the pile group is extended well out into the shoulders of the embankment fill.

The pile group layout determines the type of arching in the embankment fill and the consequent loads carried by the reinforcement. The three types of pile group layout are triangular grid layout of individual piles; square grid layout with connecting beams; and square grid layout of individual piles. The two latter layouts are the most common.

Connecting beams are sometimes used across the tops of the pile caps in the lateral direction across the width of the embankment where the foundation soil is very soft, or where the piles used cannot tolerate lateral movements. The adoption of connecting beams creates a two-dimensional arch in the embankment fill above the piles, Figure 13a. Two-dimensional arching can be analysed in plane strain with a fairly simple-geometry arch and consequent deflected reinforcement shape, Figure 13a. The resulting tension generated in the reinforcement as a result of the two-dimensional arch is relatively low, Figure 13a(iv).

The square grid pile group layout is the most common form of pile support. It also generates the most complicated arching geometry in the embankment fill, being three-dimensional, Figure 13b. Due to its three-dimensional nature, this pile group layout results in a more complicated deflected reinforcement shape, and a greater plan area of loading. Consequently, the tension generated in the reinforcement is significantly greater than for a similar geometry two-dimensional pile group layout (compare Figure 13b(iv) with Figure 13a(iv)).

6.2 *Reinforcement loads in reinforced piled embankments*

The loads generated in the reinforcements in piled embankments are due to two mechanisms. Firstly, the reinforcement acts to transfer the vertical embankment load not supported by the embankment arch to the pile caps. Secondly, the reinforcement counteracts the horizontal outward thrust of the embankment fill. The load due to arching occurs both along the length and across the width of the embankment. The load due to horizontal outward thrust occurs across the width of the embankment only.

6.2.1 *Reinforcement loads due to arching in the embankment*

Because of the relative incompressibility of the pile caps in comparison to the soft foundation, arching occurs within the embankment fill. The resulting loads in the reinforcement due to arching are dependent on the layout of the piled foundation, the height of the embankment and the nature of the embankment fill. Changes in the frictional component of the fill does not affect the amount of arching significantly but changes in the cohesive component of the fill can affect the amount of arching significantly.

Arching can be a complicated phenomenon. A number of different analysis techniques have been developed to determine the amount of arching in a piled embankment ranging from simplified analytical models, e.g. Carlsson (1987), Hewlett & Randolph (1988), BS 8006 : 1995; to small-scale physical models, e.g. Low et al. (1994), Kempfert et al. (1999); to numerical models based on continuum methods, e.g. Rogbeck et al. (1998), Kempton et al. (1998), Rogbeck et al. (2000). The results obtained have been varied.

Until recently, the use of analytical models has been the most commonly used technique. These models divide the problem into two separate parts. The first part determines the vertical stress acting on the soft foundation between the pile caps by means of a suitable arching model. The second part determines the consequent load in the reinforcement by assuming a specific deflected shape. While decoupling the problem into two separate parts simplifies the analysis, any direct interaction between soil and reinforcement is ignored.

A variety of analytical arching models are in existence and they may be divided into two groups – those modelling two-dimensional behaviour and those modelling three-dimensional behaviour. Each model is used to determine an "arching ratio", which is the ratio of the vertical stress acting on the soft foundation between the pile caps to the average vertical stress at the base of the embankment, p'_f/σ'_v.

Two well known two-dimensional arching models are shown in Figure 14. The first, by Rogbeck et al. (1998), using the triangular arching geometry devel-

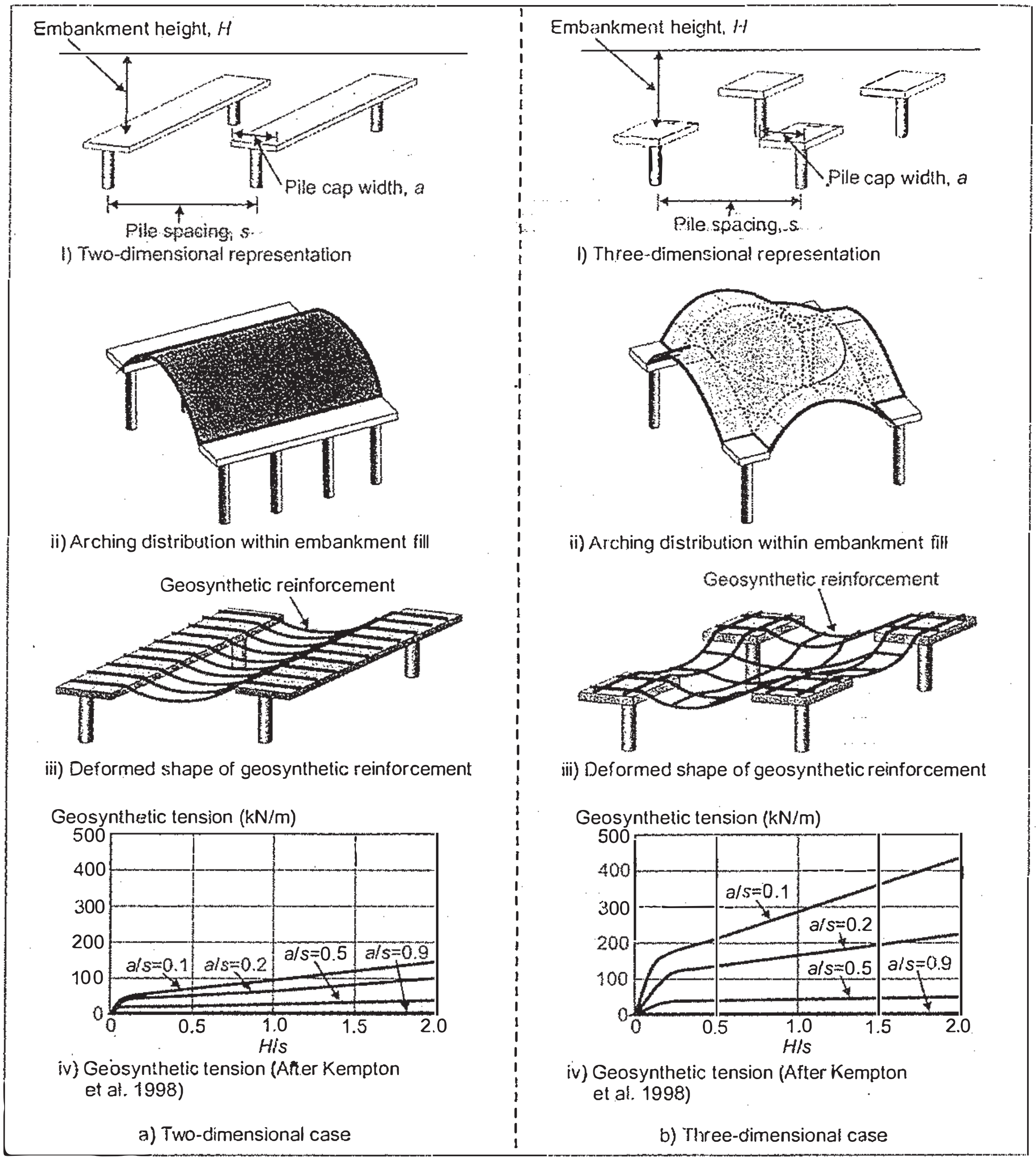

Figure 13. Effect of pile group layout on embankment arching and resulting reinforcement loads.

oped by Carlsson (1987), assumes that arching approximates a triangle within the embankment with a 30° apex angle. The resulting arching ratio relationships are plotted in Figure 14a. The use of triangular shapes of different apex angles, e.g. Jenner et al. (1998), result in arching ratios of different magnitudes, but having the same trends. The second, by Low et al. (1994), using the circular arching geometry developed by Hewlett & Randolph (1988), assumes that arching approximates a semi-circle within the embankment. The resulting arching ratio relationships are shown in Figure 14b. The differences between the two sets of curves are to be noted with the curves of Low et al. (1994) depicting greater arching.

Two well known three-dimensional arching models are shown in Figure 15. The first, by Lawson (1995) uses curve-smoothing applied to the algebraic arching equations of BS 8006 : 1995 to derive the arching ratio relationships shown in Figure 15a. The

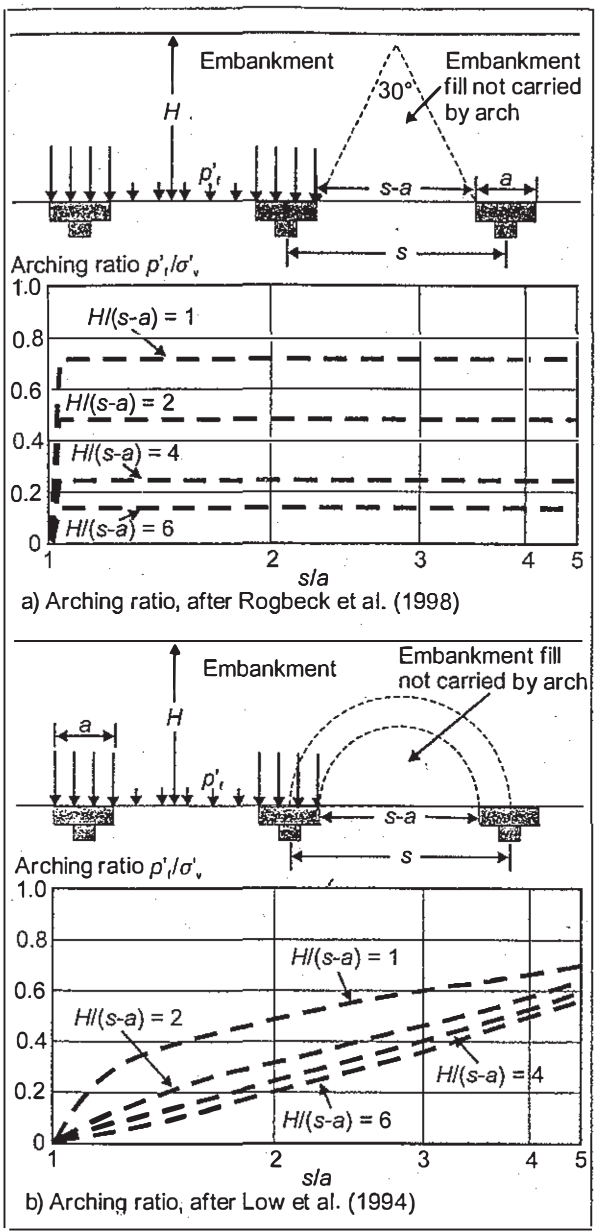

Figure 14. Two-dimensional analytical arching models.

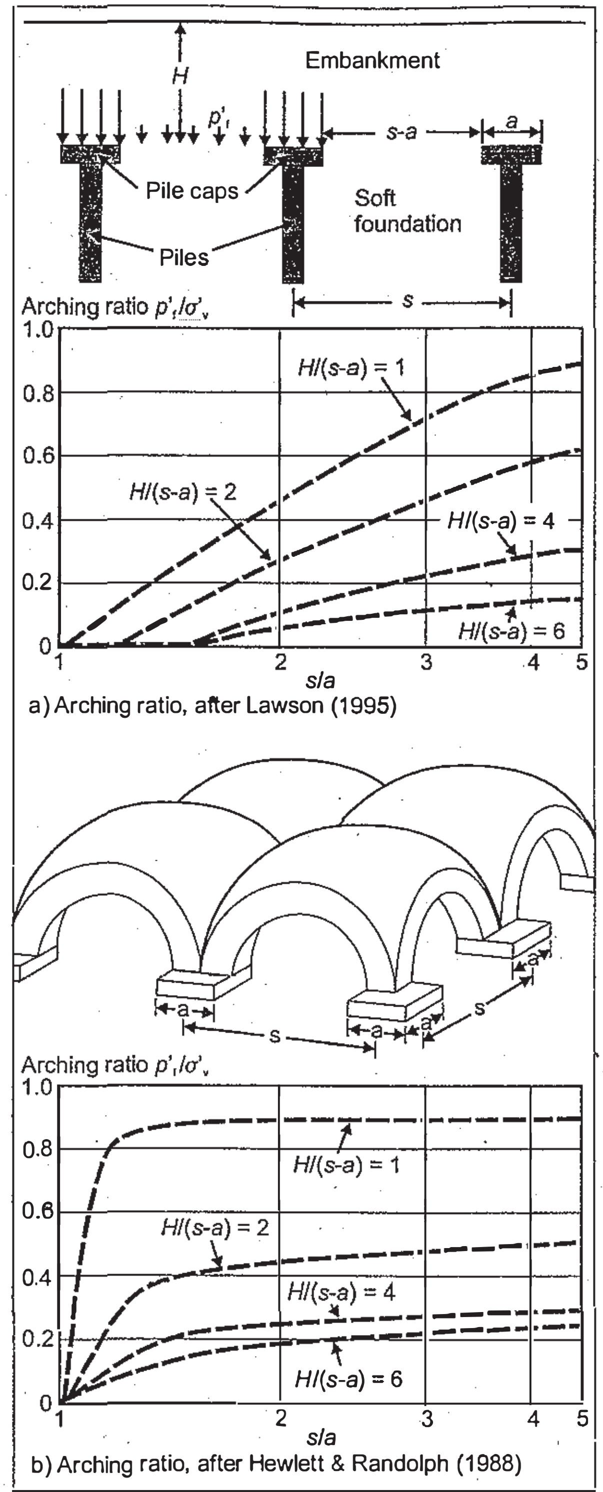

Figure 15. Three-dimensional analytical arching models.

second, by Hewlett & Randolph (1988) uses an assumed semi-spherical arching model, with the derived arching ratio relationships shown in Figure 15b. Again, the differences between the two sets of curves are to be noted with the curves of Lawson (1995) generally depicting greater arching.

More recently, the use of sophisticated numerical modelling based on continuum methods has enabled a better insight into arching and the suitability of the various analytical models, e.g. Kempton et al. (1998). In general, the semi-circular model of Low et al. (1994), Figure 14b, is in relative agreement with the two-dimensional continuum models, while the semi-spherical model of Hewlett & Randolph (1988), Figure 15b, is in relative agreement with the three-dimensional continuum models. The other analytical models show agreement with the continuum models at specific geometries only.

Once the appropriate value of arching ratio has been obtained from the analytical model the resulting load in the reinforcement is determined. This is done by use of tension membrane theory with an assumed deflected shape to describe the deformed reinforcement. The assumed deflected shape can be circular, parabolic or hyperbolic. The parabolic shape is most commonly used as it approximates more closely the true catenary (hyperbolic) shape and is mathematically simpler. The tension generated in the reinforcement deflected in the shape of a parabola is given by the following relationship;

$$T_{rp} = \frac{A_c(s-a)p'_f}{2}\sqrt{1+\frac{1}{6\varepsilon}} \qquad (2)$$

where, T_{rp} = reinforcement tension; A_c = relative coverage area of the reinforcement; s = spacing between adjacent piles; a = size of pile caps; p'_f = vertical stress acting on the soft foundation between the pile caps; and ε = strain in the deflected reinforcement.

For the two-dimensional (plane strain) case the relative coverage area of the reinforcement A_c = 1.0, however, for the three-dimensional case $A_c > 1.0$. The reason for this is that the plan area of spanning the reinforcement influenced by the pile caps is much greater for the three-dimensional case. The three-dimensional method of Lawson (1995) and BS 8006 : 1995 assumes $A_c = s/a$, while that of Rogbeck et al. (1998) assumes $A_c = 1+(s-a)/4a$. Because of the magnitude of A_c, and the three-dimensional arching model used, the loads calculated in the reinforcement are significantly greater for the three-dimensional case.

The tensile stresses in the reinforcement are at a maximum around the edges of the pile caps because it is here that the point of maximum curvature occurs.

In recent times considerable use has been made of continuum methods to determine reinforcement tensions due to embankment arching, e.g. Kempton et al. (1998), Rogbeck et al. (1998), Rogbeck et al. (2000). The advantage of continuum methods is that it is not necessary to de-couple embankment arching from reinforcement tension as is necessary in the analytical methods. Also, it is not necessary to assume a specific deflected reinforcement shape. Continuum methods can also account for the interaction between reinforcement and surrounding soil. In general, the continuum methods have shown the three-dimensional analytical methods of Rogbeck et al. (1998) and Lawson (1995) to be conservative (and hence safe). However, for an accurate appraisal of reinforcement tensions continuum methodology would appear the best approach.

6.2.2 *Reinforcement loads due to horizontal outward thrust of embankment*

The reinforcement also has to counteract the horizontal outward thrust of the embankment and it should do this at a strain level consistent with the allowable horizontal movements of the piles. It is common practice to relate this outward thrust to the maximum fill height of the embankment.

In situations where connecting beams are used across the width of the embankment then it is assumed that the connecting beams will carry the full horizontal outward thrust of the embankment.

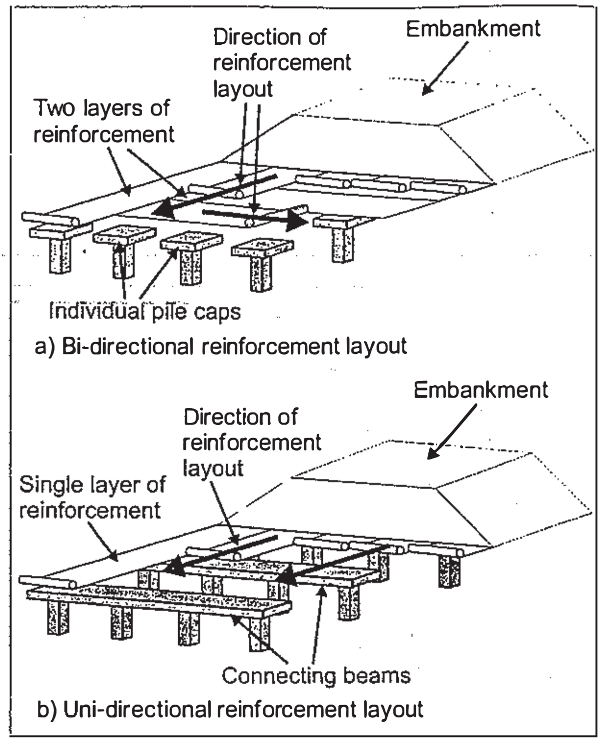

Figure 16. Reinforcement layout according to pile group geometry.

6.2.3 *Reinforcement layout to efficiently carry the tensile loads*

In reinforced piled embankments the tensile loads are generated in two directions – along the length of the embankment and across its width. The transfer of the unarched portion of the embankment loading occurs in the two directions while the transfer of the horizontal outward thrust occurs across the width of the embankment only.

The efficient layout of the reinforcement to absorb these loads depends on the type of pile cap geometry used. If the tops of the piles consist of individual pile caps then the reinforcement has to transfer the tensile loads in both the length and cross directions in the embankment. To do this efficiently it is best practice to use two layers of uni-directional reinforcement laid at right angles to each other, Figure 16a. In this case the reinforcement layers should be laid in continuous lengths in both directions to avoid joins at the base of the embankment. Where it is impossible to avoid joins along the reinforcement lengths then a minimum overlap of three times the pile group spacing should be used. Sewing and seaming techniques are not appropriate for this application.

In situations where connecting beams are used across the width of the embankment the reinforcement only has to transfer the vertical embankment loads not supported by arching in the direction right

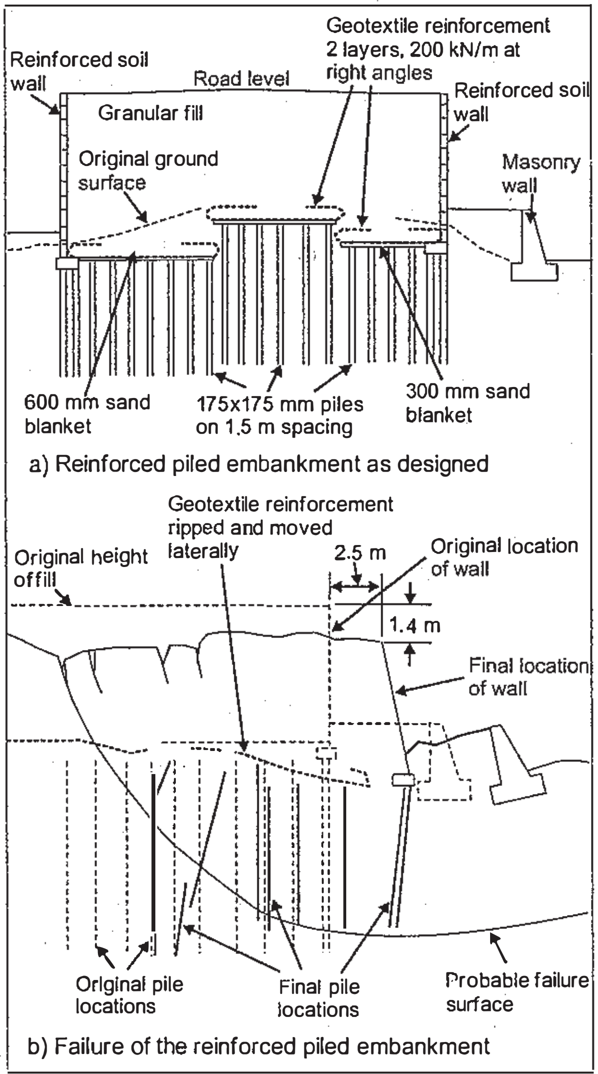

Figure 17. Example of failure of reinforced piled embankment.

angles to the connecting beams. This reduces the loading to a two-dimensional condition and a single layer of uni-directional reinforcement laid along the length of the embankment will suffice, Figure 16b. Again, where joins are inevitable, a minimum overlap of three times the pile group spacing should be used along the length of the reinforcement.

The importance of proper pile group detailing and reinforcement layout cannot be over-emphasized. Many of the problems that have arisen with piled embankments can be traced back to this. One example, shown in Figure 17, is a well-documented failure that occurred in Malaysia in 1997, Jamalludin et al. (1999), Gue & Chen (2000). The reinforced embankment was part of a highway near the new Kuala Lumpur international airport. This section of the highway passed over swampy land that had been recently filled. To prevent unacceptable settlements leading up to a bridge overpass a piled foundation was used and a double-sided reinforced soil wall adopted to minimize land acquisition costs. The reinforced soil walls ranged in height up to 11 m. The design for the reinforced piled embankment is shown in Figure 17a.

There are a number of unusual features concerning the design of this reinforced piled embankment. Firstly, the reinforcement is divided into three separate sections across the base of the embankment. It is unlikely that this layout would resist any appreciable outward thrust of the embankment fill. Secondly, a 300 mm thick sand blanket is used on top of the piles instead of pile caps. On a soft foundation it is unlikely that this sand layer would provide adequate substitution to properly designed pile caps. Thirdly, for this geometry it is unlikely that the specified required ultimate reinforcement strength of 200 kN/m would be adequate.

During construction of the reinforced soil walls, with their height between 7 to 8 m, and immediately following heavy rain, failure of one section of the embankment occurred, Figure 17b. The general mode of failure of the embankment shows the piles displacing horizontally as a slip surface forms through the embankment and into the soft foundation. The reinforced soil wall in this location moved horizontally around 2.5 m and sank around 1.4 m. The geosynthetic reinforcement in this location displaced horizontally and was extensively damaged and torn.

The results of a post-failure investigation revealed the following. First, the soft foundation soil where the failure occurred was much softer than assumed in the design. Second, the concrete piles were poorly installed and not founded in the firm stratum beneath the soft foundation layer, were poorly spliced, and were poorly finished with steel bars protruding from the tops of the piles (tearing the geotextile reinforcement). Third, because no pile caps were used the piles could not support the embankment loading and consequently much of the embankment loading was carried directly by the soft foundation. Fourth, the geosynthetic reinforcement, being discontinuous, could not provide adequate resistance to the horizontal disturbing forces. Fifth, the geosynthetic reinforcement supplied and installed (and subsequently tested) was only 100 kN/m in strength; one half of the 200 kN/m strength specified.

A re-analysis of the embankment using the arching relationships presented above shows that by not including pile caps the required reinforcement loads are very high – 650 kN/m across the embankment and 450 kN/m along the embankment. This is a considerable difference to that in the original design, and to that supplied and installed. These re-analysed reinforcement loads are considered extreme and a more practical solution would be to include pile caps on top of the piles to increase the amount of arching in the embankment fill and thereby reduce the magnitude of the loads to be carried by the reinforcement. Even if a significantly stronger reinforcement was used, and it extended continuously across the base of the embankment, it is impossible to conclude that the structure would have remained safe considering the problems that were observed with the piles during the failure investigation programme. Good pile support capability and stability are crucial to the good performance of piled embankments.

6.3 *Effect 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 which can be as much as 80%, 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 it, 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 time an equilibrium condition is reached where the deformed reinforcement supports the unarched embankment loading and negligible additional embankment loading is applied directly to the soft foundation. Thus, over time, the loading in the reinforcement increases and the loading on the soft foundation decreases. This condition is very difficult to predict with any degree of accuracy. Consequently, it would be safe, and conservative, to assume that the soft foundation soil provides negligible support for the embankment fill over the full design life of the embankment.

6.4 *Serviceability limits for low-height reinforced piled embankments*

Recently in Asia there has been considerable interest in the use of the piled embankment technique for low-height embankments. This has been motivated by the need to provide settlement-free abutments to low-height structures such as concrete culverts, and to prevent the differential settlements of services founded in low-height embankments over soft foundation soils.

Normal-height piled embankments rely on the arching occurring fully within the embankment fill and consequently, negligible differential deformations occur at the embankment surface. However, for low-height embankments the arching may not be contained fully within the embankment fill and the local vertical deformations may affect the serviceability of the embankment surface. The problem of incomplete arching and localised vertical deformations in

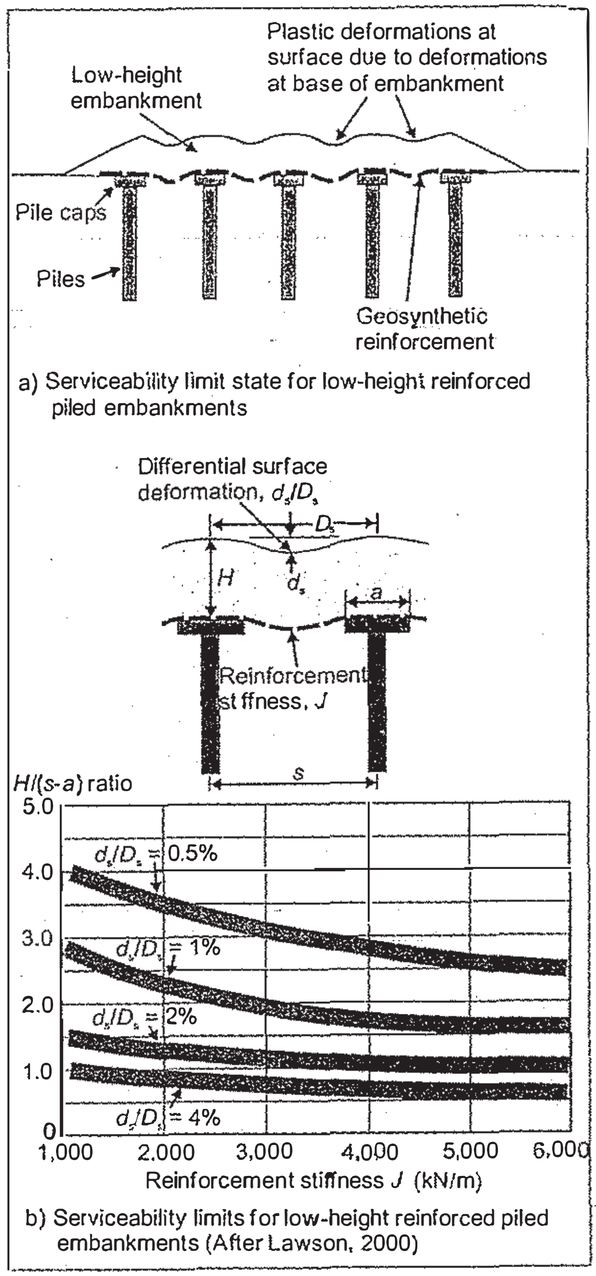

Figure 18. Serviceability limits for low-height reinforced piled embankments.

low-height reinforced piled embankments is shown in Figure 18a. The resulting differential surface deformations create problems with regard to pavement quality and riding quality.

The effect of the various reinforced piled embankment parameters on embankment serviceability is shown in Figure 18b. It is noted that specific combinations of embankment geometry (in terms of $H/(s-a)$ ratio) and reinforcement tensile stiffness are required to fulfil a specific surface serviceability requirement. For good riding quality, highways normally require $d_s/D_s \leq 1\%$.

7 PERFORMANCE RELATED ISSUES AFFECTING REINFORCED SOIL RETAINING WALLS

The vast majority of reinforced soil walls constructed in Asia have consisted of cast concrete segmental panel facings with metallic reinforcements. The reason for this is that this particular system has had a long history of use in Asia (30 years), is a well-developed system, has in most instances demonstrated excellent long-term performance, and has demonstrable economic performance benefits. The earliest example of the use of this particular system in Asia was a wall constructed in Japan in 1971. Recent evaluation of this structure has shown the good long-term performance (30 years) of the metallic strip reinforcements used, Boyd (2001).

Up until the early 1990's geosynthetic reinforced soil retaining walls were more of a novelty than a serious retaining wall technique. The reason for this was that cost effective retaining wall *systems* with the required performance utilizing geosynthetics were not commonly available. Since that time the situation has changed somewhat with the increased use of geosynthetic reinforced segmental block retaining wall systems.

Today in Asia a number of different reinforced soil retaining wall systems are used, most being proprietary in nature. The majority of these systems have been imported from Europe and North America.

7.1 *Facings for retaining wall systems*

The facing in a reinforced soil retaining wall system gives external form to the retaining wall; provides an aesthetically acceptable finish; prevents ravelling of the reinforced fill caused by weathering; provides local support to the reinforced fill between reinforcement layers, and anchors the reinforcement in the active zone. Thus, the facing must be aesthetic, structural (depending on the type of facing used) and durable.

In Asia, various facings are used in reinforced soil retaining wall systems, namely:

1 Precast concrete, segmental panels;
2 Precast concrete, full-height panels;
3 Precast concrete, segmental blocks;
4 Gabion baskets;
5 Internal wrap-around geosynthetic facings with external block facings;
6 Geosynthetic wrap-around.

Where necessary, facings should be manufactured and installed to required tolerances. This ensures a consistent facing with good aesthetics. Furthermore, the surface alignment of the facing foundation is important for the accurate seating of the facing units.

Various connections are used between the facings and the reinforcements. Precast concrete segment and full-height panels utilize positive connections with strip, mesh or geogrid reinforcements. Gabion baskets utilize positive or friction connections with mesh or geogrid reinforcements. Precast concrete, segmental blocks utilize friction, positive connection or friction and positive connections with geogrid and geotextile reinforcements.

7.2 *Reinforced fills for retaining wall systems*

In reinforced soil retaining walls the fill provides the majority of the internal shear resistance at tolerable deformations. Furthermore, the reinforced fill must enable an adequate bond to develop between the fill and reinforcements, and must not adversely affect the long-term performance of the reinforcements. These reinforced fill attributes must remain consistent over the required design life of the structure.

The factors that affect the performance of reinforced fills over time and the normal solutions adopted in their control are listed in Table 1. It should be noted from Table 1 that water can have a major effect on the performance of reinforced fills over time.

In Asia many different types of reinforced fill have been used successfully for reinforced soil walls. The choice of the appropriate fill not only depends on its mechanical and chemical properties and cost but also on local climatic conditions, in particular, rainfall.

The range of fill gradations used in reinforced soil walls in Asia is shown in Figure 19. These range from highly granular gravel materials to fine-grained sandy silts. The fill gradations can be divided into two groups – frictional and fine-frictional fills. (The term fine-frictional fill has been used here instead of the more conventional term "cohesive-frictional" because in many cases in Asia fills of these finer gradations may not possess cohesive properties.) Another important feature differentiates these two fill groups - frictional fills are essentially free draining when compacted while fine-frictional fills are not. Examples of frictional fills are fine-crushed rock, gravel sands,

Table 1. Factors affecting the performance of reinforced fills over time.

Factor	Solution
Fill soundness	Ensure fill particles do not break down
Chemical behaviour	Ensure fill has required degree of chemical inertness
Water ingress	
heavy, persistent rainfall	use free-draining frictional fill
surface runoff and groundwater	good detailing of waterproofing and drainage
hydraulic regimes	ensure fills are hydraulically stable

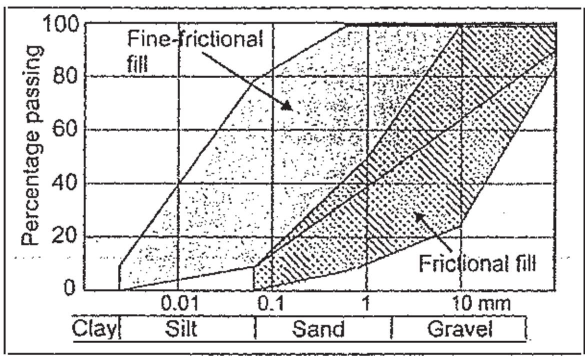

Figure 19. Range of fill gradations used in reinforced soil walls in Asia.

river sands, mining sands, crushed silica-cemented sandstone, etc. Examples of fine-frictional fills are residual and saprollytic soils, crushed clay-cemented sandstone, quarry dust, silty sands, sandy silts, loess, pulverised fly ash (PFA), etc.

In tropical, wet climates frictional fills are used exclusively for reinforced soil walls. Because of their free-draining nature, frictional fills are easy to work with between periods of heavy rainfall. Furthermore, after completion, the compacted frictional fill remains insensitive to prolonged periods of water ingress.

In dry and temperate climates a wider range of fill types may be used for the reinforced fill. However, it should be noted that as the amount of fines increase then greater attention needs to be paid to the degree of compaction and compaction uniformity. This requirement can give rise to construction difficulties, particularly near the wall face, where only limited compactive effort may be allowed in order to prevent excessive horizontal face movements.

In areas of seasonal prolonged rainfall, e.g. monsoonal climates, the use of fine frictional fills, especially those with cohesive characteristics, should be avoided unless special waterproofing means are adopted. The reasons for this are twofold. First, when the fill is placed during the "dry" season adequate compaction near the wall face may be difficult with these soil types resulting in preferential drainage paths within the reinforced fill. Second, during the "wet" season water may easily penetrate these preferential drainage paths resulting in a loss of shear resistance in the fine-frictional fill and a loss of reinforcement bond resistance.

7.3 *Reinforcements for retaining wall systems*

A number of different reinforcement types are used in reinforced soil walls in Asia, namely:

1 Metallic strips of galvanised steel;
2 Metallic grids and meshes of galvanised steel;
3 Polymeric strips of polyester and polypropylene;
4 Geogrids of polyester, high density polyethylene, glass and aramid;
5 Geotextiles of polyester and polypropylene.

Formal approval procedures exist for reinforcements in the more regulated countries of Australia, Japan and Hong Kong. In the less regulated countries reliance is placed on the designer or manufacturer to specify and provide the appropriate reinforcements. In the poorly regulated countries the initial tensile strength of the reinforcements is normally the only selection criterion.

Over recent years there has been much more confidence in the long-term performance of the reinforcements. This has quite correctly lead to some relaxation in the reinforcement strengths used. However, the quick adoption of lower strength reinforcements should be approached with some care as they may show greater susceptibility to harmful effects, such as installation damage, than the stronger reinforcements.

7.4 *Deformations in reinforced soil retaining walls*

The paramount performance related issue affecting reinforced soil retaining walls in Asia is that of deformation. Deformations occur in reinforced soil walls for a variety of reasons, namely:

1 During construction, deformations may occur because of the construction procedure adopted.
2 Failure to fulfil in practice the ultimate limit states determined in design.
3 The geometry of the reinforced soil mass.
4 Volume changes and deformations that may occur in the reinforced fill over time.
5 Excessive deformations that may occur in the reinforcements over time.
6 External applied loads.
7 Face flexibility.
8 Compressibility of the foundation.

In practice, formal design codes normally stipulate an allowable amount of deformation for reinforced soil walls for a given function. For example, BS 8006 : 1995 stipulates vertical and horizontal deformation limits for acceptable serviceability. In general, these deformation limits have been extracted from design methods for other retaining wall systems, such as cantilever walls, etc., which could be interpreted as unduly stringent given reinforced soil's flexibility. However, there is some rationale behind this (more) restrictive approach inasmuch as a client is not concerned with what type of retaining wall he gets as long as it performs the required function. Historically, this has been provided by less flexible structures, e.g. cantilever or gravity walls, and they have provided acceptable performance, hence the more stringent deformation requirements.

Acceptable deformation limits should be established according to the function of the retaining wall, and the framework set out in Figure 4 can be used as a rational basis for this. Where reinforced soil walls are to be used in conjunction with associated structures, e.g. pavements, services, load supports, etc., then the deformation limits of the associated structures need to be considered when establishing the Stage 1 (Figure 4) deformation limits for the retaining walls. Where no associated structures are involved then more relaxed limits approximating the Stage 2 (Figure 4) deformation limits may be warranted.

While there are eight reasons for deformations in retaining walls listed above, it is proposed to highlight the effect of only two of these in this paper. The first deals with the effect of the geometry of the reinforced mass on deformations, and the specific aspect covered will be trapezoidal reinforcement sections. The second deals with the effect of compressible foundations. Both of these aspects have become major performance related issues with reinforced soil walls in Asia in recent years.

7.4.1 *Deformations associated with retaining walls having trapezoidal reinforcement sections*

As the move to reduce the cost of reinforced soil retaining walls has intensified over the last few years, more emphasis has been placed on the use of reinforced soil walls with trapezoidal reinforcement sections. The reason for this is that trapezoidal section walls have considerable economies compared to conventional reinforced soil wall geometries in terms of reduced excavation and fill quantities and reduced quantity of reinforcement. Comparison between a conventional rectangular geometry reinforced soil wall and a trapezoidal section wall is shown in Figures 20a and 20b.

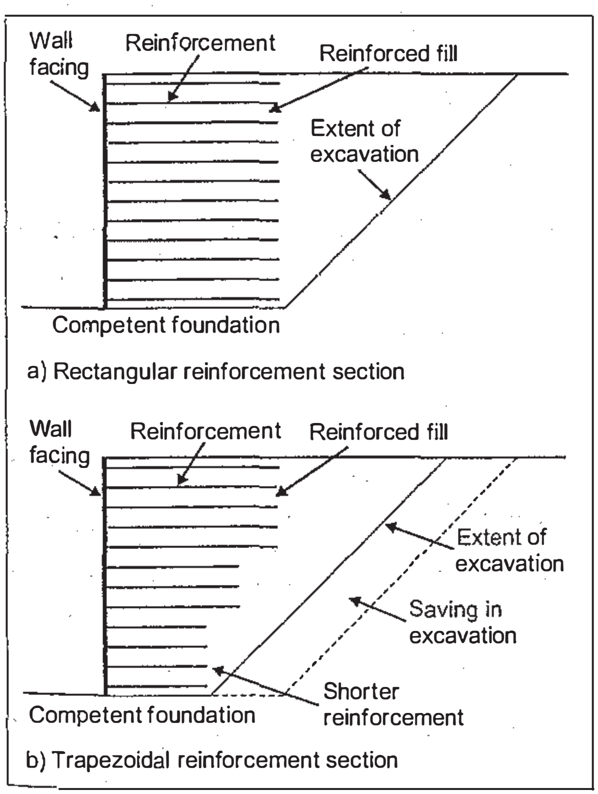

Figure 20. Trapezoidal-section wall showing economies compared to rectangular section wall.

Trapezoidal-section walls may be used where there is a competent foundation to support the increased vertical stresses at the base of the wall. In some design codes, e.g. BS 8006 : 1995, it is inferred that the type of foundation beneath a trapezoidal-section wall should be limited to a rock or rock-like foundation. However, in reality, the foundation only has to be competent enough to ensure excess vertical deformations do not arise as a result of the increased stresses due to the changed geometry of the trapezoidal wall. Thus, trapezoidal-section walls also may be constructed safely on top of overconsolidated foundation soils that demonstrate small deformations under the imposed retaining wall loads.

In some instances the cost of a trapezoidal-section wall may be half the cost of a conventional, rectangular, reinforced soil wall. This provides much of the impetus behind the increased use of trapezoidal-section reinforced soil retaining walls.

An intrinsic feature of trapezoidal-section walls is that their mode of horizontal movement during and after construction is different compared to rectangular-section walls. Figure 21a shows the difference in movement of these two types of walls using the same reinforcement and reinforced fill characteristics. The rectangular-section wall shows the conventional increasing horizontal movement with wall height. This mode of movement is normally compensated for during construction to ensure the completed wall has a near-vertical alignment. The trapezoidal-section wall shows quite a different horizontal movement profile with height, with the maximum horizontal movement occurring between one quarter and one half of the wall height. In addition, this maximum movement can be twice the maximum horizontal movement in a rectangular-section wall when the same materials are used.

It is impossible to fully compensate for the horizontal movements of trapezoidal-section walls during construction alone. The judicial adoption of fill type, reinforcement characteristics and trapezoidal-section geometry can be utilized to minimize the horizontal movement differences between trapezoidal-section and rectangular-section walls, Figure 21b. Once this is done, the wall can be constructed by compensating for the conventional "ideal" horizontal movements normally associated with rectangular-section walls.

The magnitude of the horizontal movements associated with trapezoidal-section walls is dependent on the sliding and pull-out characteristics of the lower half of the reinforced mass. These are governed by the height of the wall, the length of the reinforced zone at the base of the wall and the bond coefficient between the reinforced fill and the reinforcements. The bond coefficient has two components; reinforced fill shear

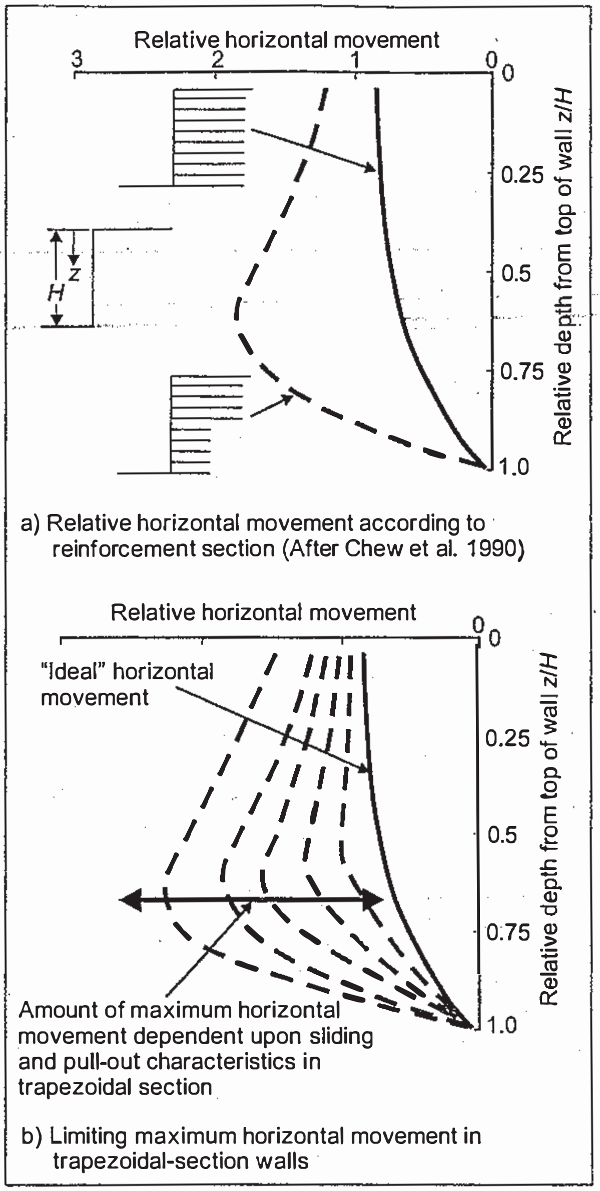

Figure 21. Mode of horizontal deformation of a trapezoidal-section wall compared to a rectangular section wall.

resistance and reinforcement interaction coefficient, and these are related by:

$$\mu_{s/GSY} = f_{s/GSY} \tan\phi' \tag{3}$$

where $\mu_{s/GSY}$ = bond coefficient between reinforced fill and reinforcements; $f_{s/GSY}$ = reinforcement interaction coefficient; and ϕ' = shear resistance of the reinforced fill. Thus, it is necessary to obtain suitable combinations of reinforced fill shear resistance and reinforcement interaction coefficient to ensure the bond coefficient requirement is satisfied. Failure to meet this requirement will result in undue horizontal movements around mid-height in the reinforced soil wall.

Figure 22 shows the results of wedge analyses to determine the required bond coefficient between reinforced fill and reinforcements in trapezoidal-section walls at the ultimate limit state. Conventional trapezoidal-section geometries were utilized in the analysis, Figure 22a. In addition, the effect of reinforcement surface area coverage is also taken into consideration in the form of a b/a ratio value, Figure 22b. To generate the curves shown in Figure 22c ultimate limit state partial factors recommended by BS 8006 : 1995 are used. Thus, the curves can be considered to give safe solutions.

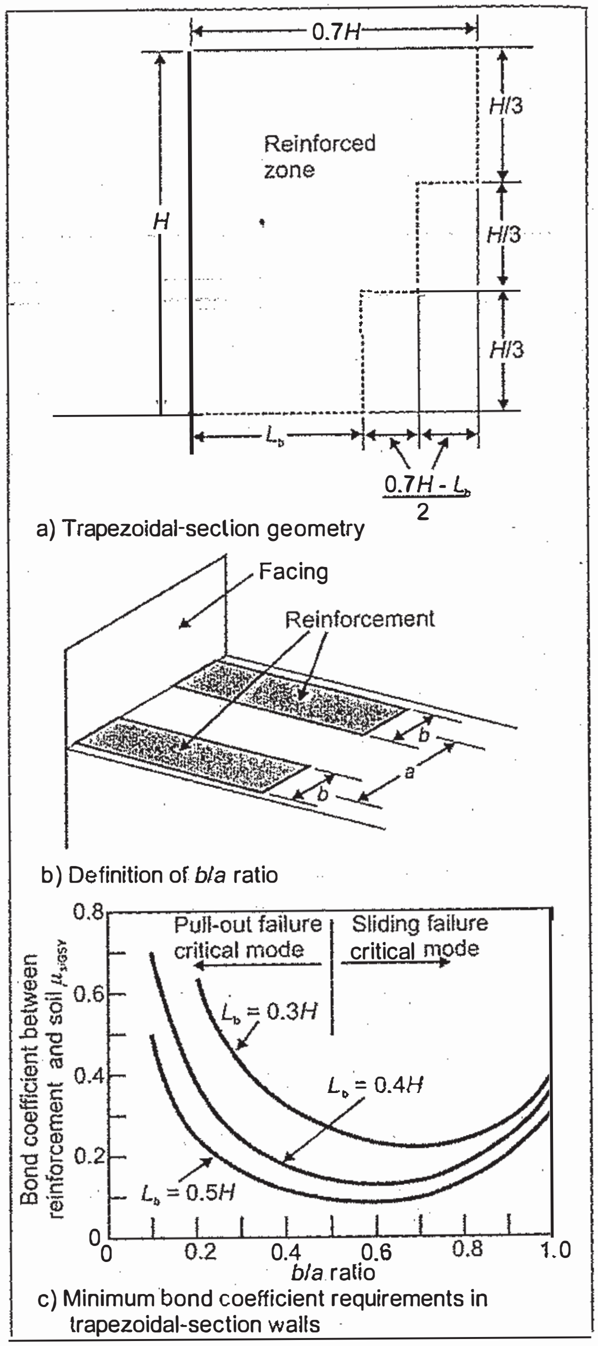

Figure 22. Required minimum reinforcement bond coefficient for trapezoidal-section walls at the ultimate limit state.

The results in Figure 22c show a demarcation in failure mode, and hence cause of deformation, around a reinforcement b/a ratio of 0.5. For reinforcements with b/a ratios less than 0.5 the critical failure mode, and cause of deformation, is pull-out, whereas with b/a ratios greater than 0.5 the critical failure mode, and cause of deformation, is sliding. The results also show that greater bond coefficients are required where the base length of the reinforcement L_b is shorter and where the reinforcements are narrower. The results also show that the lowest bond coefficients are required where the reinforcement b/a ratio is approximately 0.5 to 0.7. Furthermore, full coverage, planar, reinforcements, i.e. b/a ratio = 1.0, require a lower bond coefficient for stability than narrow strip reinforcements but do not provide the lowest values required.

Equation 3 can be used to determine the required values of fill shear resistance and reinforcement interaction coefficient to meet the required values of bond coefficient from Figure 22c. This will maintain the horizontal deformations in trapezoidal-section walls within tolerable limits.

7.4.2 *Deformations associated with the presence of compressible foundations*

With its large-scale coastal developments it is inevitable that many infrastructure projects in Asia are constructed in poor ground conditions. In coastal and low-lying areas many problems arise because of low bearing strength and compressible foundation soils. Because of their flexibility and their ability to perform in environments where significant deformations occur, reinforced soil retaining structures are used in these environments, e.g. Schodts (1990).

Compressible foundations present a number of problems for retaining walls, most notably post construction deformations. Figure 23 shows the various types of deformations that can occur to retaining walls due to the presence of a compressible foundation. The two fundamental modes of wall movement resulting from the presence of a compressible foundation are a vertical movement downwards as a result of the weight of the retained fill behind the wall and a horizontal movement inwards (into the wall) as a result of the differential vertical deformation behind and in front of the base of the retained fill behind the wall. These two fundamental modes of wall movement give rise to all of the deformation conditions shown in figure 23.

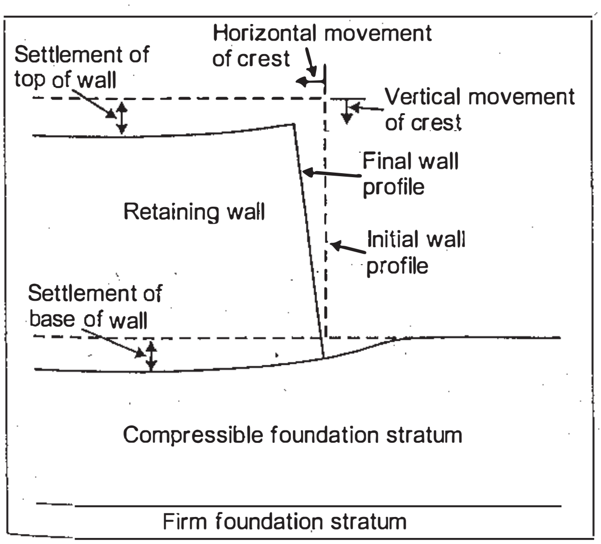

Figure 23. Types of retaining wall deformations arising from compressible foundations.

To ensure that retaining wall deformations remain within acceptable limits, treatment is normally applied to the compressible foundation. The treatments vary, ranging from soil replacement, to piling, to foundation consolidation techniques. It is convenient to divide the approach adopted according to the depth of the soft foundation layer.

7.4.2.1 Compressible foundations of shallow depth

Compressible foundations of shallow depth, say to a maximum of 3 m, can cause significant deformations in retaining structures. The geometry of the problem is shown in figure 24 where a retaining wall of height H is situated on a compressible foundation that has an overconsolidated surface crust of 0.5 m to 1 m in thickness. The two modes of wall deformation of interest with regard to retaining wall serviceability are the vertical and horizontal movements of the wall crest. These are designated as ΔY and ΔX respectively in figure 24, with the positive direction being upwards in the vertical direction and outwards in the horizontal direction.

Figure 24 also shows the results of analyses using continuum methods to determine the relative magnitudes of the vertical and horizontal wall crest movements due to the presence of a 2.5 m thick compressible foundation. The results are plotted in terms of a ratio of the crest movement to the wall height versus the elastic modulus of the compressible foundation layer. Negative crest movement ratios indicate a vertical relative movement downward (i.e. vertical settlement) and a horizontal relative movement inward. As is to be expected, the relative movements in both the vertical and horizontal directions decrease with increasing foundation stiffness. It is important to note that the relative vertical movement is approximately an order of magnitude greater than the relative horizontal movement; the relative horizontal movement being small.

Where compressible foundations are of shallow depth then often the most economical technique to limit deformations is to excavate the compressible foundation soil and replace it with compacted fill. To gain the maximum benefit excavation should be carried out down to a firm stratum. Figure 25 shows the effect of foundation soil replacement on the relative movements of the crest of the retaining wall. The re-

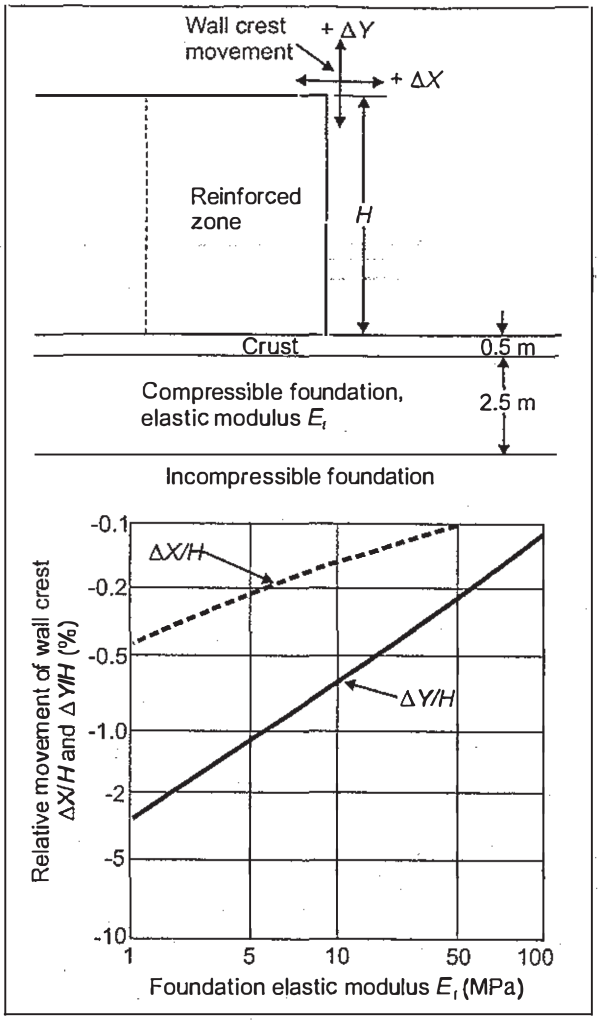

Figure 24. Vertical and horizontal movements of wall crest due to foundation stiffness.

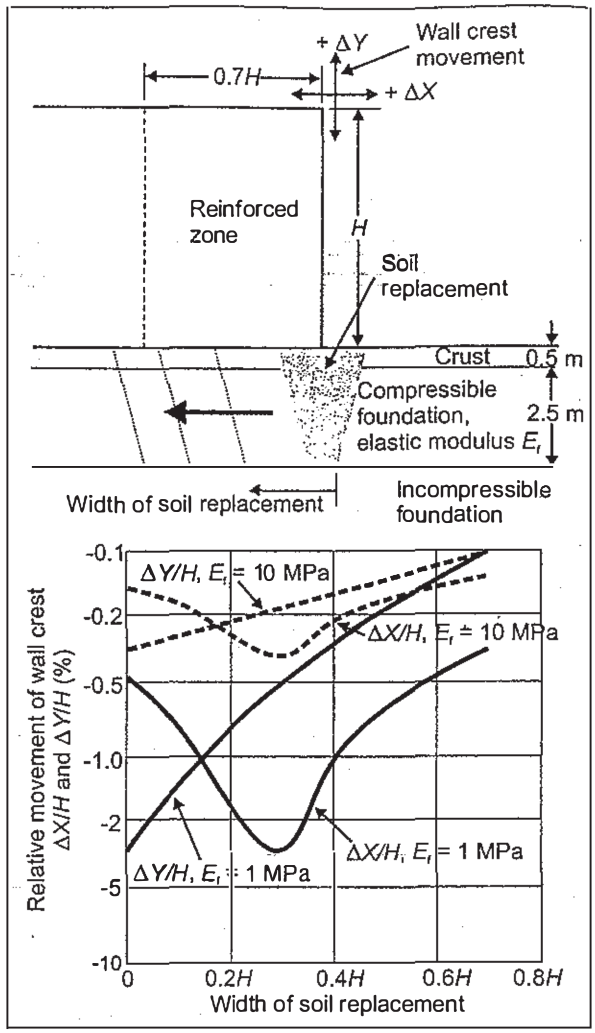

Figure 25. Effect of foundation soil replacement on vertical and horizontal movements at the crest of reinforced soil walls.

sults are plotted comparing relative wall crest movement with width of soil replacement across the base of the wall and existing compressible foundation stiffness. It is observed that the relative vertical crest movement decreases significantly with only limited width of soil replacement. However, the relative horizontal crest movement *increases* for soil replacement widths up to $0.3H$, and then decreases to a minimum at $0.7H$ (the extent of the reinforced zone in the retaining wall). Thus, while partial replacement of the compressible foundation beneath a reinforced soil wall may significantly reduce the vertical movement of the wall crest it may actually *increase* the horizontal movement with the soil replacement acting as an unyielding fulcrum. To minimize *both* vertical and horizontal wall crest movements the foundation replacement should extend beneath the whole width of the reinforced zone.

The example shown in Figure 26 is a reinforced segmental block wall constructed on a compressible foundation layer. The wall was 5.5 m in height and was founded on a 0.75 m thick overconsolidated crust overlying a 2 m thick compressible foundation layer. Because of the requirement to control the vertical movement of the wall crest the compressible foundation layer was excavated below the wall face and replaced with compacted fill of width approximately 1.5 m.

On completion of the wall it was observed that while the vertical movement of the wall crest was small, and within tolerances, the wall was rotating inwards by a significant amount. The curves plotted in Figure 25 were used to estimate what the maximum inward rotation would be. Using the geometry shown in Figure 26 and the compressibility characteristics of the soft foundation layer a maximum inward rotation ($\Delta X/H$) of 3% could be expected over time. This amounts to a horizontal inward movement of 0.17 m. The wall movements were monitored for a period of one year. After eight months the wall movements reached equilibrium with a maximum inward movement of 0.15 m, which is close to the predicted movement. The vertical movement of the wall crest of 0.02 m also agrees closely with the predicted value.

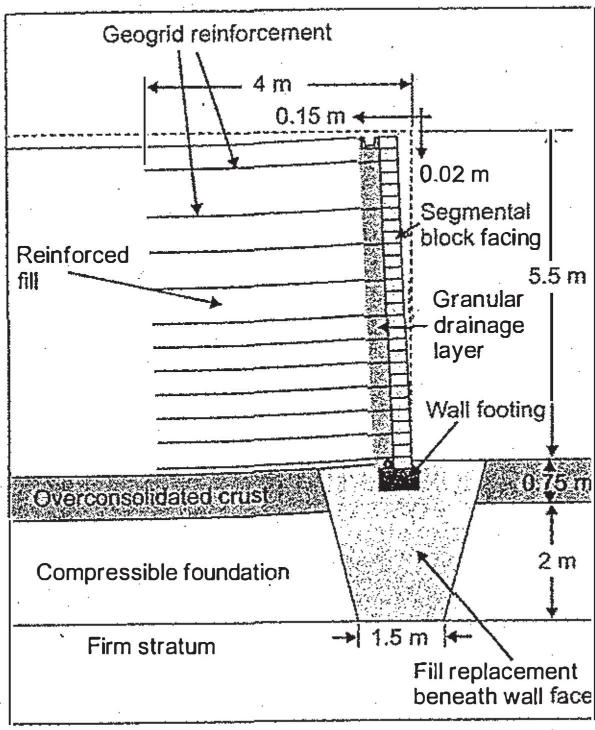

Figure 26. Reinforced soil wall founded on compressible foundation.

7.4.2.2 Compressible foundations of significant depth
Where the compressible foundation is of significant depth it is generally uneconomic to carry out soil replacement and hence insitu forms of foundation treatment are used. These normally involve pre-consolidation of the soft foundation or methods that involve transfer of the retaining wall loads onto a deeper, stronger foundation stratum. These latter methods include piling (concrete and timber piles), stone columns, grout-injected stone columns, concrete columns, jet-grouted columns, lime columns, etc. The foundation improvement measures should cover the whole area of the reinforced soil zone, including the zone of influence, in order to eliminate both vertical and horizontal movements. Smith (1990) gives a general overview of these methods in relation to reinforced soil structures.

7.5 *Reinforced segmental block retaining walls*

First used in Asia in the early 1990's, reinforced segmental block retaining wall systems provide a very economical means of constructing retaining walls. The technology is imported from North America, and much of the developments today still tend to be North American driven. The general layout of a reinforced segmental block wall is shown in Figure 27.

A major attribute of reinforced segmental block walls is their ease of construction. Low-technology construction methods can be applied which makes for low cost construction. The only mechanised equipment that is necessary is the compaction equipment for the reinforced fill. In countries where manual labour costs are low, retaining walls constructed using this technique are very cost-effective.

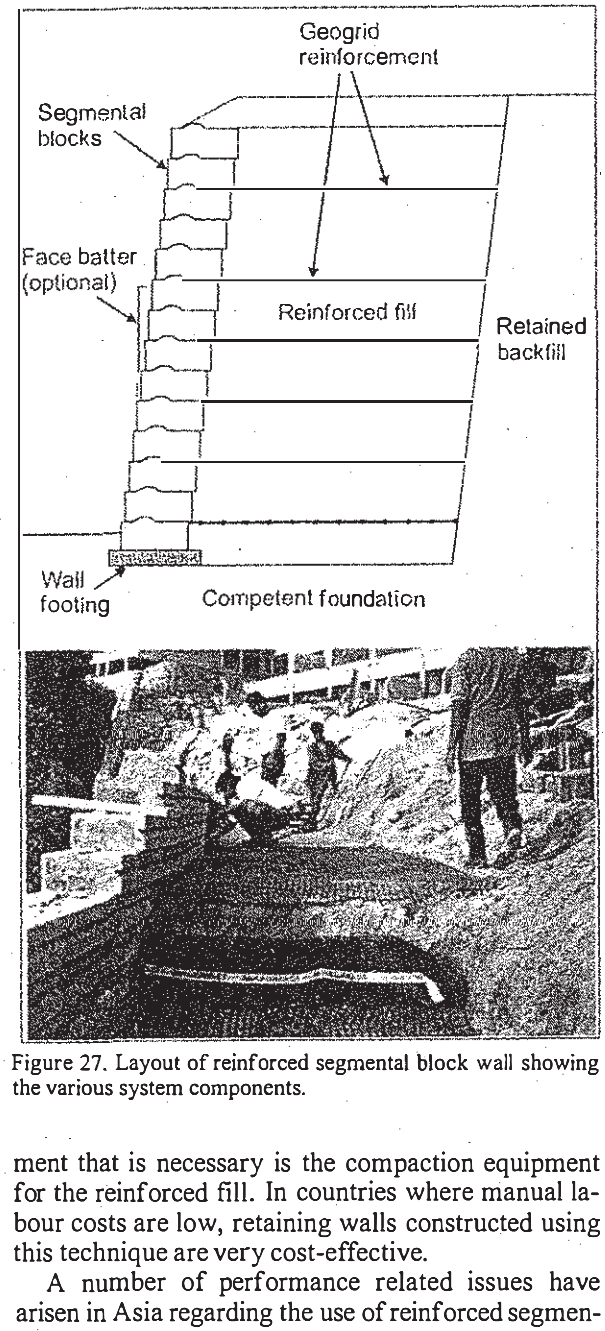

Figure 27. Layout of reinforced segmental block wall showing the various system components.

A number of performance related issues have arisen in Asia regarding the use of reinforced segmental block retaining walls. These are summarised below.

7.5.1 *Segmental wall design methodology*

As stated above, reinforced segmental wall technology was imported into Asia from North America. Consequently, North American design methodologies have been proposed for the design of these walls, e.g. NCMA (1995). This relatively recent introduction has tended to conflict with more mature reinforced soil design methodologies originally derived from European practice. This has resulted in some confusion regarding the merits of segmental block walls, and some suspicion.

In an attempt to rationalise these differences, major government organizations in the more developed reinforced soil countries of Australia and New Zealand have modified the design methods to conform with existing reinforced soil retaining wall design codes. These modifications have inevitably followed the most conservative design route. This has resulted in a two-tier approach to reinforced segmental block wall design in these countries. For projects controlled by these major government organizations their specific design procedure is used, however, for other projects less-conservative design methods, e.g. NCMA (1995), are used as these give the most cost-effective solutions.

7.5.2 *Segmental blocks*
Most of the segmental blocks used in Asia are based on proprietary technology. Much of this technology revolves around the type of connection between adjacent blocks and the type of connection between blocks and the reinforcements. To improve stability of the block facing various block connection technologies are employed ranging from simple friction to different-shaped shear keys to shear pins.

To ensure an even vertical stress distribution within the block facing the blocks must be manufactured to specific dimensions and tolerances. The flatter and wider the block the more severe the dimensional tolerances have to be. The blocks also have to be limited in weight in order to be easily installed by manual labour.

7.5.3 *Wall footing*
The wall footing has an important role in segmental block retaining walls. In addition to providing a stable toe for the retaining wall it also establishes the initial grade line for the block wall. The grade line needs to have a uniform finish to ensure the blocks are well-seated, and needs to adjust appropriately for changes in grade levels, etc. While the wall footing can consist of compacted fill in many cases it is composed of concrete in order to obtain the smooth and consistent grade line necessary.

7.5.4 *Reinforcement/block connections*
Reinforcement/block connections range from simple friction to partial friction/partial positive connection to full positive connection.

Theoretically, the tensile stress carried in the reinforcement at the rear of the wall face is fairly low and therefore the connection capacity between the reinforcement and the block facing is small. However, because of differential movements between block facing and the reinforced fill, and compaction stresses, greater loads are carried at the connections than are revealed by theory. Furthermore, external loads in the vicinity of the wall face can increase the connection loads significantly. Thus, connection capacity between the segmental blocks and the reinforcement is more important than would initially be assumed.

In areas of potential extreme external loading, e.g. seismic areas, it would appear that block systems utilising full positive connections offer the best solution.

7.5.5 *Reinforcements*
There are two families of reinforcements used in reinforced segmental block walls in Asia, namely:

1 Geosynthetic reinforcements, of which the major variety is geogrids, and
2 Metallic ladder-like reinforcements composed of galvanised steel.

The vast majority of reinforced segmental block walls in Asia are constructed using geogrid reinforcement. A differentiation of use is developing between geogrid reinforcement and metallic ladder-like reinforcement inasmuch as the metallic ladder-like reinforcement appears to be preferred where low strain levels are required. An example of this would be a bridge abutment where horizontal deformations in the reinforced soil mass are to be minimized. For all other applications geogrid reinforcement appears to be preferred because of its cost-effectiveness.

The comments made in Section 7.3 regarding the reinforcements in reinforced soil retaining walls in general also pertain to reinforced segmental block walls.

7.5.6 *Reinforced fill*
The comments on reinforced fills in Section 7.2 also pertain to reinforced segmental block walls.

7.5.7 *Drainage*
It has to be recognised that drainage is an integral part of reinforced soil wall design especially in wet climates. If frictional fill is not used as the reinforced fill in the retaining wall then drainage measures need to be included in areas of groundwater and surface water activity. Subsurface drainage layers are required behind and beneath the reinforced zone to intercept groundwater before it enters the reinforced fill. Surface drainage measures are also required at the wall crest to capture surface run-off in a controlled manner.

Drainage detailing is also important for these retaining walls. Well designed and installed drainage exit points at the base of the wall face should be an integral component of these walls.

7.5.8 *Wall construction quality*
The major economy associated with reinforced segmental block walls is their ease of construction. Because construction utilizes low technology construction procedures a whole new generation of installers are constructing these walls. It is unfortunate that many of the lessons learned from the construction of the more mature reinforced soil wall

systems have not been passed on readily to this newer technique. For example, it is well known amongst existing reinforced soil wall contractors that the wall face moves outwards during construction of a reinforced soil wall, and that allowances are made for this during construction. Unfortunately, many reinforced segmental block walls have been constructed without taking this intrinsic horizontal movement into account. Consequently, these walls do not exhibit smooth, vertical alignments.

In some respects it has now become "too easy" to construct reinforced soil walls. For example, an installer who is used to constructing 1 to 2 m high retaining walls can now easily construct 5 to 8 m high retaining walls utilising the same materials and similar procedures. To a fair extent, this ease of construction has come at the expense of construction quality. One way of improving construction quality would be to have a register of qualified installers. However, this approach would only be successful in the more regulated countries.

It is to be expected that over time construction quality will improve to the standard and consistency required. Certainly, there are many good quality reinforced segmental block walls constructed in Asia.

7.5.9 *Aesthetics*

A major attribute of segmental block walls is their aesthetics. Blocks can be manufactured in a variety of surface textures and colours to blend in or contrast with the surrounding landscape. One major aspect though is the maintenance of the surface finish. Fungus growth and staining of the block surface over time detracts from the aesthetics of the blocks. Fungus growth can be prevented or minimized by incorporation of fungicides in the block concrete mix. Staining may be prevented by incorporating a drainage layer behind the block facing thereby preventing groundwater from seeping through the block face. A surface interception drain at the wall crest can prevent staining due to overflow of surface run-off.

8 PERFORMANCE RELATED ISSUES AFFECTING REINFORCED FILL SLOPES

In their modern form, reinforced fill slopes have been used in Japan since the 1970's, Fukuoka (1988). The earlier reinforced slopes were constructed using woven or nonwoven geotextile reinforcements, or geonets, but since the late 1980's the vast majority of reinforced slopes have been constructed using geogrid reinforcement.

To the author's knowledge the earliest practical reinforced slope in the region outside Japan was constructed in Australia in 1980. The reinforced slope was constructed to widen a racetrack in Melbourne and was 4 m in height and had a slope angle of 3 to 1, Figure 28. The reinforcement used was a woven poly-

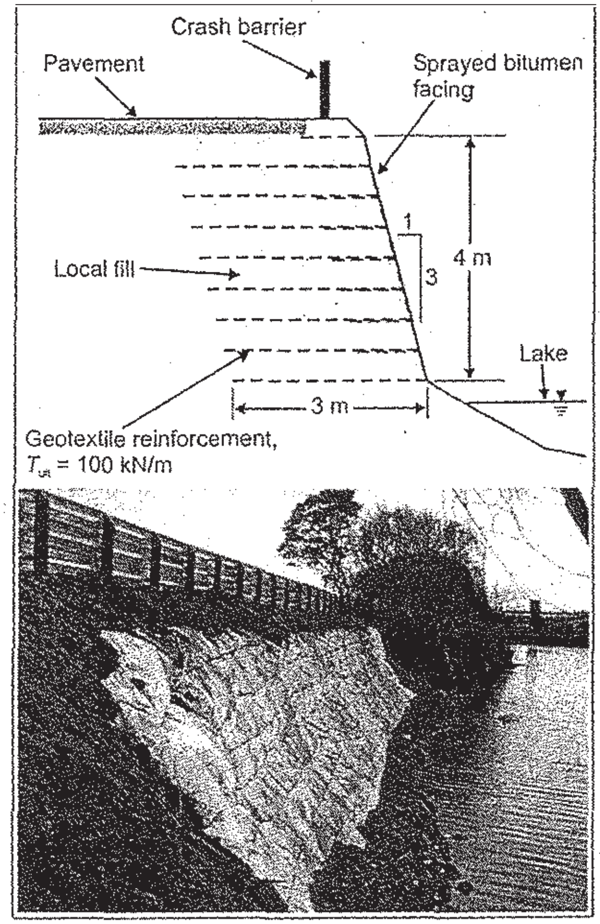

Figure 28. Reinforced slope, Melbourne, Australia, 1980.

ester geotextile with ultimate tensile strength 100 kN/m. A sprayed bitumen coating was applied to the wrap-around facing for permanent protection.

Reinforced fill slopes can be differentiated on the basis of slope angle and a simple classification is shown in Figure 29. Steep slopes are defined as those where the tension in the reinforcements has not dissipated on reaching the slope face and hence an "active", i.e. structural, facing is required. Shallow slopes are defined as those where the tension in the reinforcements has dissipated before reaching the slope face and hence a "passive", i.e. non-structural, facing is required. The region of $\beta \approx 45°$ provides the approximate slope angle transition between steep and shallow slopes.

A number of Codes of Practice differentiate between the design methodology for reinforced soil retaining walls and reinforced slopes, e.g. BS 8006 : 1995 and GCO (1989), with retaining wall design methodology extending from 90° to 70° face angle with the horizontal. The reason for this has more to do with the history of development of the design methodologies than with their efficacy of use. Retaining wall design methodologies have been based on the use of Rankine or Coulomb pressure distributions while

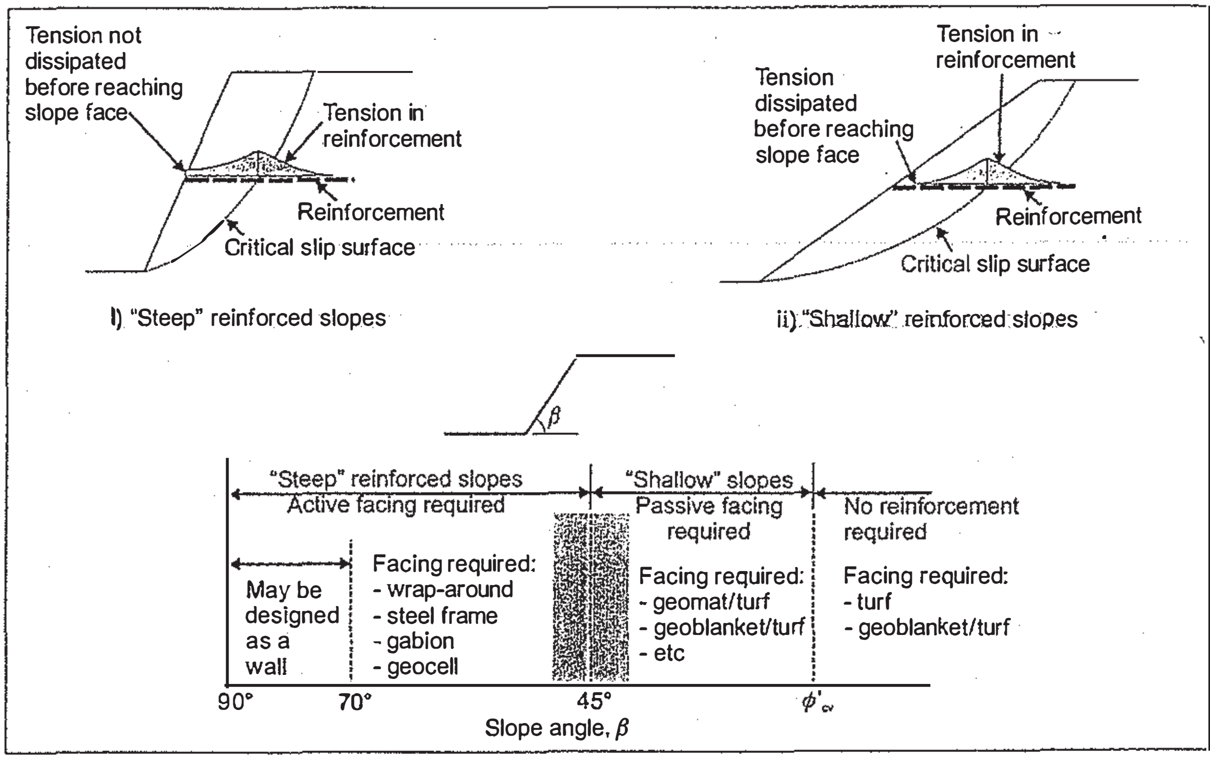

Figure 29. Classification of slopes according to slope angle.

slope design methodologies have been based on slip surface analyses.

Historically, the use of 70° as the dividing point between wall design and slope design was also due to the approach taken to these types of structures. Slopes up to an angle approaching 70° could be rendered stable with the addition of a light surface covering, however, for slopes over 70° a structural engineering solution in the form of a retaining wall was required. This retaining wall structure consisted of grouted stone, reinforced concrete, steel, or mass gravity units, and was designed by a structural engineer.

With the advent of reinforced soil as a solution for slope and wall design no longer is the 70° slope angle a defining point in design methodology as the same design approach can be employed for reinforced slopes for all slope angles almost to the vertical. The only defining point regarding slope angle in reinforced fill slope design is around 45° where the nature of the slope facing changes – the defining point between shallow and steep slopes.

8.1 *Soil types used as reinforced fills for slopes*

Much of the cost-effectiveness of reinforced fill slopes involves the use of the local soil as the reinforced fill. In Asia this has meant the widespread use of soils with significant percentages of fines. These soil types will be referred to as "fine-frictional" fill for the same reasons as stated in Section 7.2. These fine-frictional fills range from relatively uniform, granular, clay-cemented sandstone to very well-graded residual soils to "cohesive" deposits of partially-weathered shales and clay-gravels. This covers a very wide range in terms of soil gradation. However, when compacted properly all of these fine-frictional fills have adequate shear strength, with the worst exhibiting $\phi' \approx 30°$. Many of these soils have compacted $\phi'_p \geq 35°$.

The prime feature affecting the performance of these fine-frictional fills in reinforced slopes is the degree of compaction attained and the consistency of compaction throughout the slope. Good and consistent compaction of these fills is essential from the viewpoint of maximizing shear resistance and minimizing ingress of water.

In a tropic, wet environment it is difficult in practice to place and compact in a consistent manner the finer gradation soils. Rainfall during construction results in pore water pressures within the "compacted" fill thereby reducing its effective shear strength and requiring greater quantities of reinforcement, both in terms of vertical spacing and length, to achieve required stability. This is not normally considered at the design stage.

Poor compaction quality also enables easy ingress of water into the fills following construction. Water may enter by way of surface infiltration and groundwater flow. Water ingress results in soil erosion and the development of pore water pressures within the fill. As stated already, pore water pressure reduces the shear strength of the fill, but it can also decrease the strength parameters associated with soil softening. The control of soil erosion is best carried out by good attention to slope facing and surface drainage details. The control of pore water pressure build-ups is best carried out by the extensive use of subsurface drainage (see Section 8.4 below). Indeed, when dealing with reinforced slopes containing fine-frictional fills drainage is just as important as the reinforcement.

8.2 *Reinforcements used in reinforced slopes*

The earliest reinforced fill slopes used woven and nonwoven geotextiles as the reinforcements, however, since the late 1980's geogrids have become the preferred reinforcement material.

Geosynthetic reinforcements provide the ideal combination of attributes for reinforced fill slopes – tensile strength and stiffness, durability and good soil/geosynthetic bond. Ease of installation is also a major advantage.

In recent times, composite geosynthetics that combine the functions of reinforcement and drainage have been proposed as the reinforcements for fine-grained fills in reinforced slopes, e.g. Kamon et al. (1994), Chew & Loke (1996), Kempton et al. (2000). These composite geosynthetics range in structure from simple reinforced nonwoven geotextiles to fairly complex free-flowing drainage channels integrated into geogrids. The claimed benefits of using these materials are that any pore pressures existing in the reinforced fill can be quickly dissipated thereby increasing the strength of the fill in the vicinity of the drain, and improving the bond characteristics between reinforcement and fill.

What still has to be established is the required drainage performance over time of these materials. If the composite material is only required to drain excess pore pressures in the fill for a short period of time during and following construction then the drainage performance requirements are fairly rudimentary. If, however, the composite material is required to drain effectively for a considerable period of time then close attention will have to be paid to the long-term filter and drainage (and reinforcement) performance of the material for specific reinforced fill types.

These materials look to have interesting possibilities, however, it is to be emphasized that in no way should these materials be viewed as substitutes for conventional subsurface drainage measures in reinforced fill slopes, Section 8.4.

8.3 *The importance of slope facings in Asia*

Slope facings are an integral component of reinforced slopes and, depending on the steepness of the slope, can play an active, i.e. structural, role or a passive, i.e. non-structural, role, Figure 29. Furthermore, the slope facing constitutes the visual component of the reinforced slope and consequently must meet additional criteria in terms of surface erosion resistance and aesthetics. For aesthetics, increasing emphasis has been placed on the use of "green" facings.

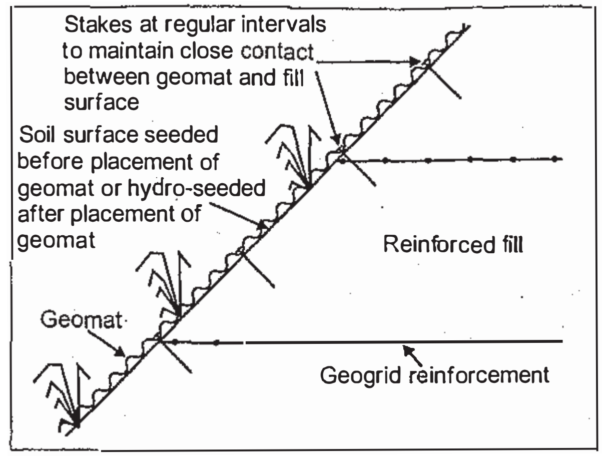

Figure 30. Vegetated facing detail for a shallow reinforced slope.

8.3.1 *Shallow reinforced slopes*

The facings of shallow reinforced slopes have no structural role in the performance of the slope but must prevent surface erosion and provide a good aesthetic appearance. These facings normally consist of a simple vegetated surface on the slope, Figure 30.

For shallow slopes the reinforcement is normally terminated at the slope face or is laid partially down the slope face. In dry, or seasonal, climates grass sodding may be used to provide the growing medium for the vegetated slope. The vegetation will have time to grow before heavy rainfall occurs. For tropical, wet climates a geomat is placed on the slope surface and attached securely to the slope as shown in Figure 30. The geomat acts to bind the root matter of the surface vegetation and prevent surface erosion of the slope.

Where geomats are used it is important that intimate contact be maintained between geomat and the exposed soil surface. This facilitates vegetation growth and prevents localised erosion beneath the geomat. Good connections at close intervals between the geomat and the soil surface of the slope are important. Furthermore, if possible, the geomat should also be connected to the extremities of the reinforcement layers at the slope face.

In some parts of Asia 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 then cut back smooth with the slope face. The compacted soil face is then covered with a geomat to prevent surface erosion and is vegetated as soon as possible.

8.3.2 *Steep reinforced slopes*

The facings of steep reinforced slopes perform a structural role as well as provide a durable, aesthetic surface. Thus, they need to be relatively substantial in

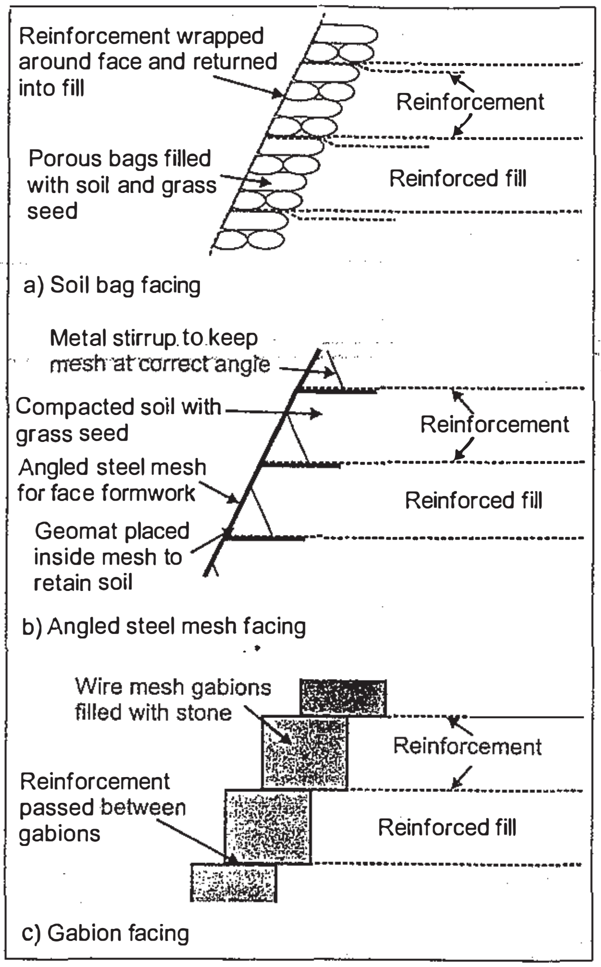

Figure 31. Three common facing techniques for steep reinforced slopes in Asia.

size compared to the passive facings for shallow slopes. Structurally, the facing needs to be able to dissipate the residual tensile stresses occurring in the reinforcement at the slope face as well as confine the reinforced fill to enable compaction near the slope face.

Three commonly used facings for steep slopes are shown in Figure 31. These are soil bag facings, angled steel mesh facings and gabion facings. The soil bag facing, Figure 31a, is the most common and consists of soil-filled bags, normally with vegetation seed mix included, placed to form the slope face. The reinforcement is wrapped around the outside of the soil bags and brought back into the slope to provide adequate local bond length. Once placed, it is possible to reshape the soil bags with an excavator bucket to provide a good quality slope finish. The technique is very cost effective and a good quality, green face is obtained.

The angled steel mesh facing, Figure 31b, consists of steel mesh, prefabricated to the required slope angle, used as a semi-rigid external formwork. The angled steel mesh elements are placed along the extremity of the slope with the reinforcement truncated at the inside base of the angled mesh. To prevent surface erosion and support vegetation growth a geomat is placed down the inside of the face of the angled steel mesh prior to placement, and light compaction, of a soil mixed with grass seed. Behind this front soil layer the reinforced fill is placed and compacted. The angled steel mesh remains in place permanently.

The gabion facing, Figure 31c, consists of wire mesh cages filled with stone as the facing units. The reinforcement can be either physically attached to the gabion cages or passed between adjacent rows of gabion cages. It is possible to have a green finish with gabion facings by including soil and grass seed with the stone in the gabion cages.

8.4 *The importance of drainage in wet environments*

Good drainage should be an integral part of reinforced fill slopes. The effective use of drainage enables groundwater and surface water to be contained and redirected before it can enter the reinforced slope and cause damage. 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 32.

The main function of the subsurface drainage system is to collect groundwater at the rear of the slope and convey it in a controlled manner to exit the slope face. The groundwater collection area for the subsurface drainage system should be founded

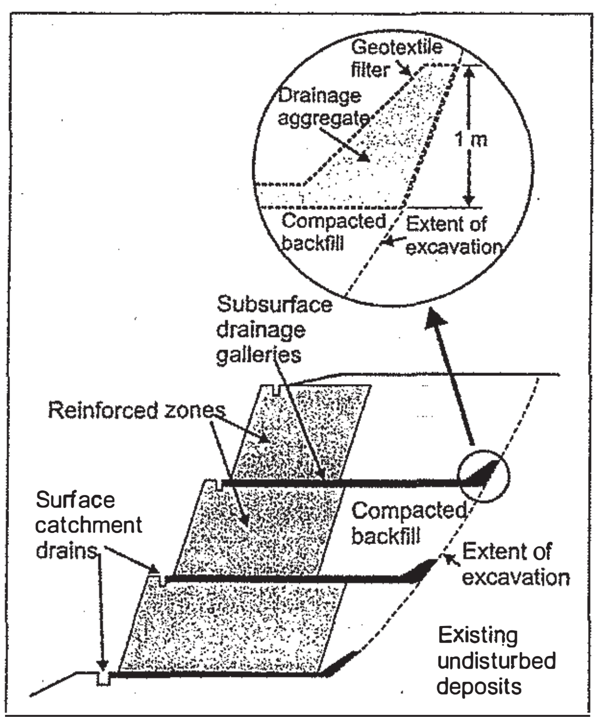

Figure 32. Recommended drainage layout in steep, high, reinforced slopes (After Lawson & Yee, 1998).

against the undisturbed part of the excavation area at the rear of the slope with enough exposed surface area to readily collect any groundwater flows. This prevents any pore pressure build-ups occurring behind the lower-permeability, compacted backfill. Well-designed drainage networks should then convey any groundwater to the exit points at the face of the slope.

The choice of the correct materials for the subsurface drains is also important for their long-term performance. Well-selected geotextile filters with the appropriate aggregates and/or pipe drains are critical.

Surface catchment drains should be installed along the crest of each tier of the reinforced slope and at the top of the slope crest. The functions of these catchment drains are to gather surface run-off and conduct it away from the slope in a controlled manner; to prevent excess surface run-off from cascading uncontrollably down the face of the slope; and to provide controlled exit points for subsurface drains.

For steep, high, reinforced slopes constructed in a wet environment the proposed drainage system should be well-planned before construction commences.

8.5 *"Standard" versus "non-standard" reinforced slope geometries*

In designing reinforced slopes a differentiation needs to be made between what is termed a "standard" reinforced slope geometry and what is termed a "non-standard" reinforced slope geometry. Standard slope geometries have relatively simple geometries and loading profiles, Figure 33a, and are normally constructed in fairly flat to mildly undulating terrain. With the emphasis placed on internal stability, these standard geometry, reinforced slopes can utilize simple, straight-forward design procedures such as chart methods, e.g. Jewell et al. (1985), Schmertman et al. (1987), Jewell (1990), and spreadsheet design methods. These simple design methods make standard geometry slopes very easy to design for, with most of the work taken up in detailing.

Conversely, non-standard slopes can have relatively complex geometries, material characteristics and loading regimes, Figure 33b. Global and compound stability issues dominate the design of these types of slopes. Consequently, non-standard slope geometries require not only more rigorous analysis procedures but also a more detailed understanding of the slope environment. It has been demonstrated that it is incorrect to apply the simple, standard slope design procedures to these more complex slope geometries, Lawson & Yee (1998).

In Asia, a significant amount of development is moving away from the crowded flat coastal plains and into the more hilly and mountainous hinterland. The earthworks associated with this development can be significant, and the adoption of reinforced fill slopes of heights over 30 m is not uncommon. Also, some of these regions are subject to severe seismic events. These conditions make it essential that a rigorous design procedure be used for these types of slopes. Furthermore, experienced engineers should carry out this design.

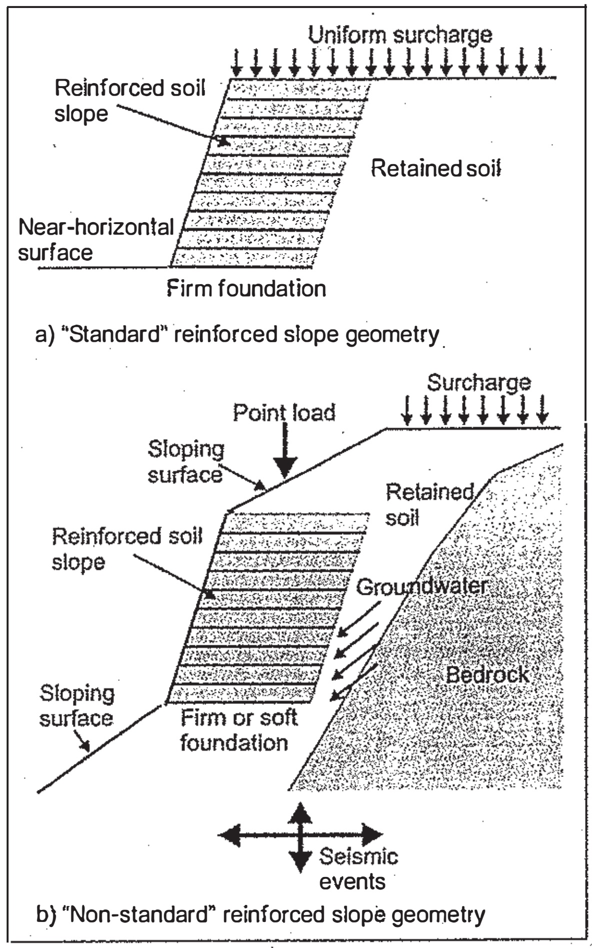

Figure 33. Standard and non-standard reinforced slope geometries.

8.6 *The impetus to minimize reinforcement lengths in reinforced slopes in Asia*

When constructing reinforced slopes in undulating terrain the base of the reinforced slope is normally founded within a cut section of the existing soil slope. In Asia, there has been an impetus toward minimizing the width of the base of the reinforced slope in order to minimize the cost of the excavation. Another reason for reduced reinforcement base width is the occurrence of harder rock material in the vicinity of the cut face. In some instances this has lead to extreme reinforced zone geometries. The construction procedure followed is shown in Figure 34.

The goal is to maximize the development area on top of the reinforced slope while minimizing the excavation and hence obtain the maximum cost-benefit, Figure 34a. The existing ground is excavated to con-

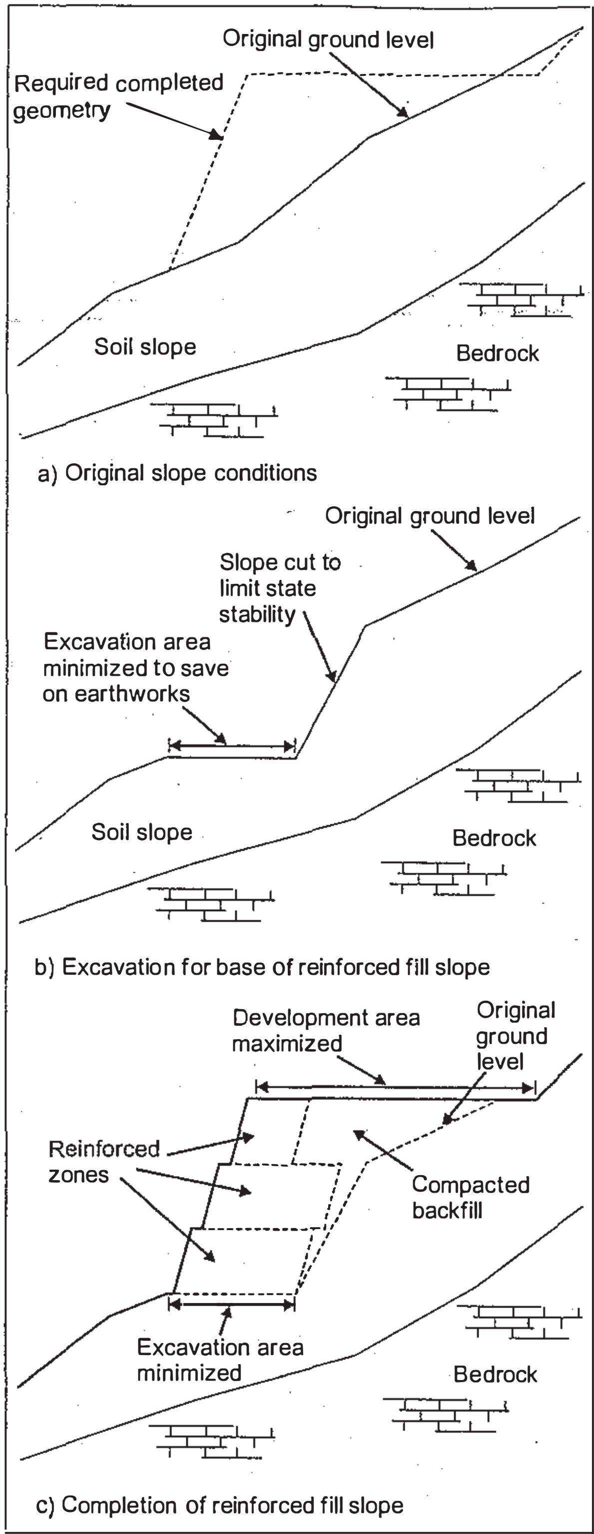

Figure 34. Common reinforced slope construction geometry in Asia.

struct the base of the reinforced slope. The excavated slope at the rear of the excavation is maintained at or near its limit state for stability. This ensures minimum excavation quantities, Figure 34b. In most instances there is little prior planning or design/analysis associated with the stability of the excavated slope. If it is not possible to provide the required excavated base width for the reinforced slope because of instability of the rear slope it is common for the reinforcements in the bottom tier of the slope to be truncated accordingly. If this occurs then it is unusual for a redesign of the slope to be done.

The completed reinforced slope geometry is shown in Figure 34c.

8.7 *The importance of competent foundations for high slopes*

The conventional design of reinforced fill slopes assumes that the foundation is competent and incompressible. As such, the capacity of the foundation does not influence the design of the reinforced fill slope. However, there are many situations where reinforced fill slopes are required and the foundation conditions do not fit the definition of "competent and incompressible".

Many of the reinforced fill slopes in Asia are constructed in hilly and mountainous terrain where the geomorphology is similar to that shown in Figure 33b. In this situation external stability governs the dimensions of the reinforced zones and the strength and compressibility of the foundation can have a significant effect. The foundation may be soft and compressible by reason of groundwater levels and unconsolidated insitu soil deposits.

Continuum methods have been used to model the effect of foundation strength and compressibility on the dimensions of the reinforced zone for the slope geometry shown in Figure 35a. The results plotted in Figures 35b and 35c are an extension of the work of Lawson & Yee (1998) where two foundation conditions have been analysed – a "firm" foundation and a "moderate" foundation. As observed, the effect of the two types of foundation conditions on the allowable dimensions of a reinforced slope is significant. For example, a 30 m high slope with a 70° slope angle would require a 10 m base width on the firm foundation and would require a 24 m base width on the moderate foundation in order to satisfy external stability conditions.

In general, if the foundation beneath a reinforced slope is unconsolidated and/or affected by groundwater it is very difficult to improve it by means of compaction. Furthermore, improvement in the compressibility and shear resistance of the foundation requires compaction for some depth beneath the base of the reinforced zone. This can prove expensive and may even be impossible if significant quantities of groundwater are present. If these conditions occur, in most situations, it is better to make allowances for foundation compressibility and shear resistance in the design of the reinforced slope rather than attempt to modify the properties of the foundation.

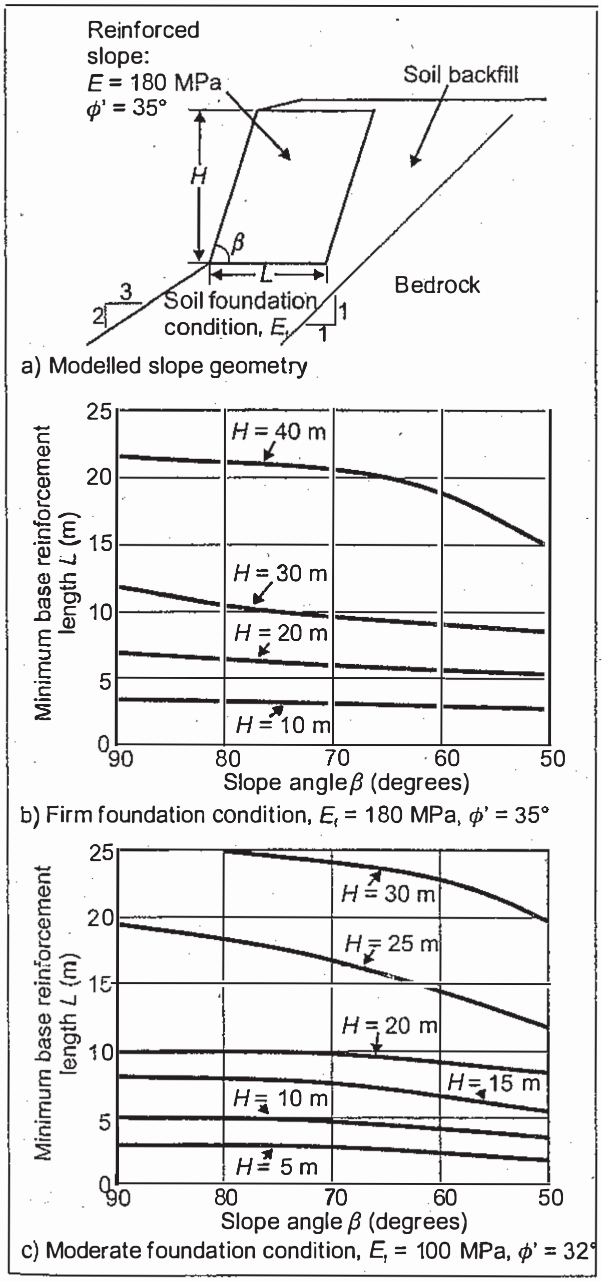

Figure 35. Effect of foundation conditions on the minimum reinforced slope base dimensions required for external stability.

8.8 *Effect of combinations of the above performance related issues on reinforced slopes*

While the performance related issues affecting reinforced fill slopes have been discussed separately, in practice, these normally occur in combinations. To illustrate how these combinations can affect the performance of a reinforced slope a case study is given below.

A reinforced slope solution was proposed to enable road access into a hill-side development site at Linkou, Taiwan in 1993. The existing topography was sloping ground consisting of uncompacted colluvium and dumped clay-gravel spoil overlying bedrock. The original design consisted of a reinforced slope of 20 m height with an average slope angle of 60°. The slope was to consist of 4 reinforced fill tiers with a reinforcement length of 12 m at the base of the slope. Details of the original design layout are shown in Figure 36a.

As excavation into the loose ground progressed slope instability at the rear of the excavation became a major problem to the extent that it was impossible to maintain the width of excavation at the base of the reinforced slope to the required 12 m. Consequently, it was decided to adopt a reduced width of 7 m for the base of the reinforced slope with all other design details remaining unchanged. The revised layout of the constructed slope is shown in Figure 36b.

It was also impossible to densify the loose foundation soil beneath the reinforced slope to any appreciable depth. This, in combination with the reduced reinforcement lengths at the base of the reinforced slope, significantly reduced the external stability of the slope.

Considerable groundwater flows were observed from the site even during the dry season when construction was carried out. Consequently, extensive subsurface drainage galleries were included at the base of each reinforced soil tier. The galleries extended to the rear of the excavation at each level.

The reinforced slope utilized a wrap-around facing using soil bags, Figure 31a. The local excavated soil was used as the compacted reinforced fill in the slope and geogrids were used as the reinforcement. Thc construction quality of the reinforced slope was very good.

Immediately following completion of the third reinforced tier a tension crack appeared along the rear of the top reinforced zone. Over the next day this tension crack increased in width to around 3 cm. Concurrently, it was observed that considerable horizontal movements were occurring at the toe of the slope – around 0.5 – 0.7 m. It was evident that the slope had become unstable and urgent corrective measures were required in order to prevent possible failure. Subsequent modelling of the reinforced slope using continuum methods was able to reproduce the instability with close approximation to the observed deformations, Figure 36c.

To prevent further instability it was decided to construct a toe berm to buttress the bottom tier of the reinforced slope. The details of this toe berm are shown in Figure 36d. The local soil was used in the toe berm and it was constructed to the height of the first reinforced tier. During construction of the toe berm subsurface drainage outlets at the base of the reinforced slope were extended out beyond the toe berm. Once the toe berm had been completed the tension crack behind the reinforced zone at the top of the third tier was filled in and sealed. Following placement of

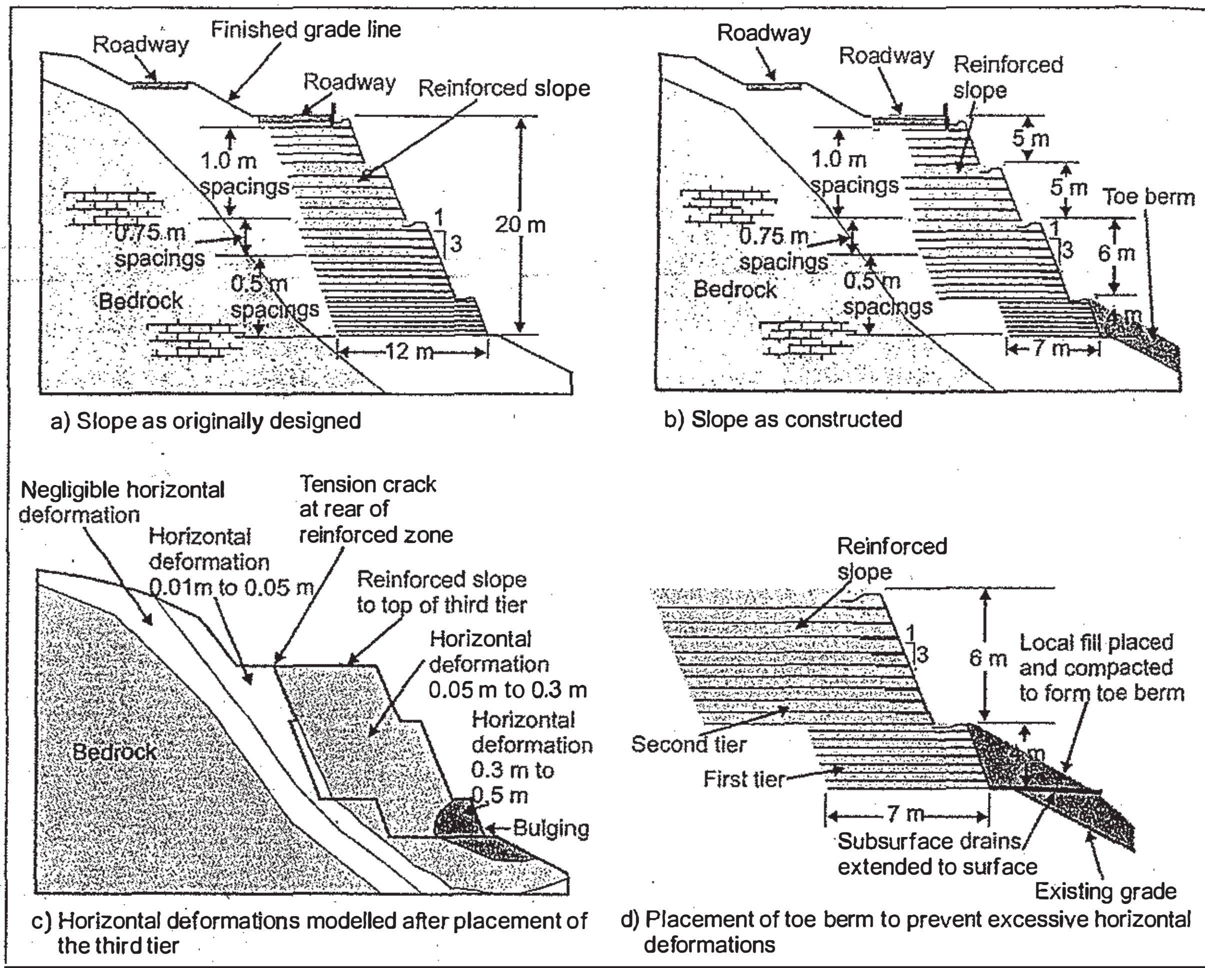

Figure 36. Reinforced fill slope, Linkou, Taiwan.

the toe berm no further movement of the slope was observed. Subsequent modelling of the reinforced slope, with the toe berm in place, was able to demonstrate that the toe berm was very effective in maintaining the stability and preventing further deformation in the corrected reinforced slope. It must be stated that quick action by the contractor in placing the toe berm prevented a possible costly failure of the reinforced slope.

The fourth reinforced tier and the roadways were then constructed, Figure 36b.

Over 6 years following construction the reinforced slope was periodically monitored by visual inspections. No further deformations or instability have been observed. The vegetation has completely overgrown the slope face. The roadway pavement that is situated directly over the rear of the reinforced zone has shown no sign of cracking or deformation either within or near the pavement structure (cracking could be expected in the pavement if the reinforced slope was moving). The drainage systems continue to drain water from the slope, with the subsurface water being clear with no sediment.

8.9 *Reinforced slopes under extreme loading conditions*

Parts of Asia are subject to severe seismic events. Accounting for seismic activity requires longer reinforced zones and closer reinforcement spacings. Well-designed and constructed reinforced fill slopes have demonstrated good performance during severe seismic events, e.g. Tatsuoka et al. (1996).

Very rarely do reinforced fill slopes collapse on a significant scale, and it needs a combination of unusual reinforced zone geometry and an unusual event for this to occur. More often, localised collapse or excessive movements may happen. When a large collapse occurs the lessons can be highly instructive. The example given below concerns the collapse of a reinforced fill slope in Taiwan during the famous Chichi earthquake of 1999. An analysis of this collapse also has been published elsewhere, Holtz et al. (2001).

The reinforced fill slope was situated at the entrance to Chi-Nan University, near Puli in central Taiwan, and served as part of the slope construction measures carried out to construct the main access entrance into the University. The reinforced slope was

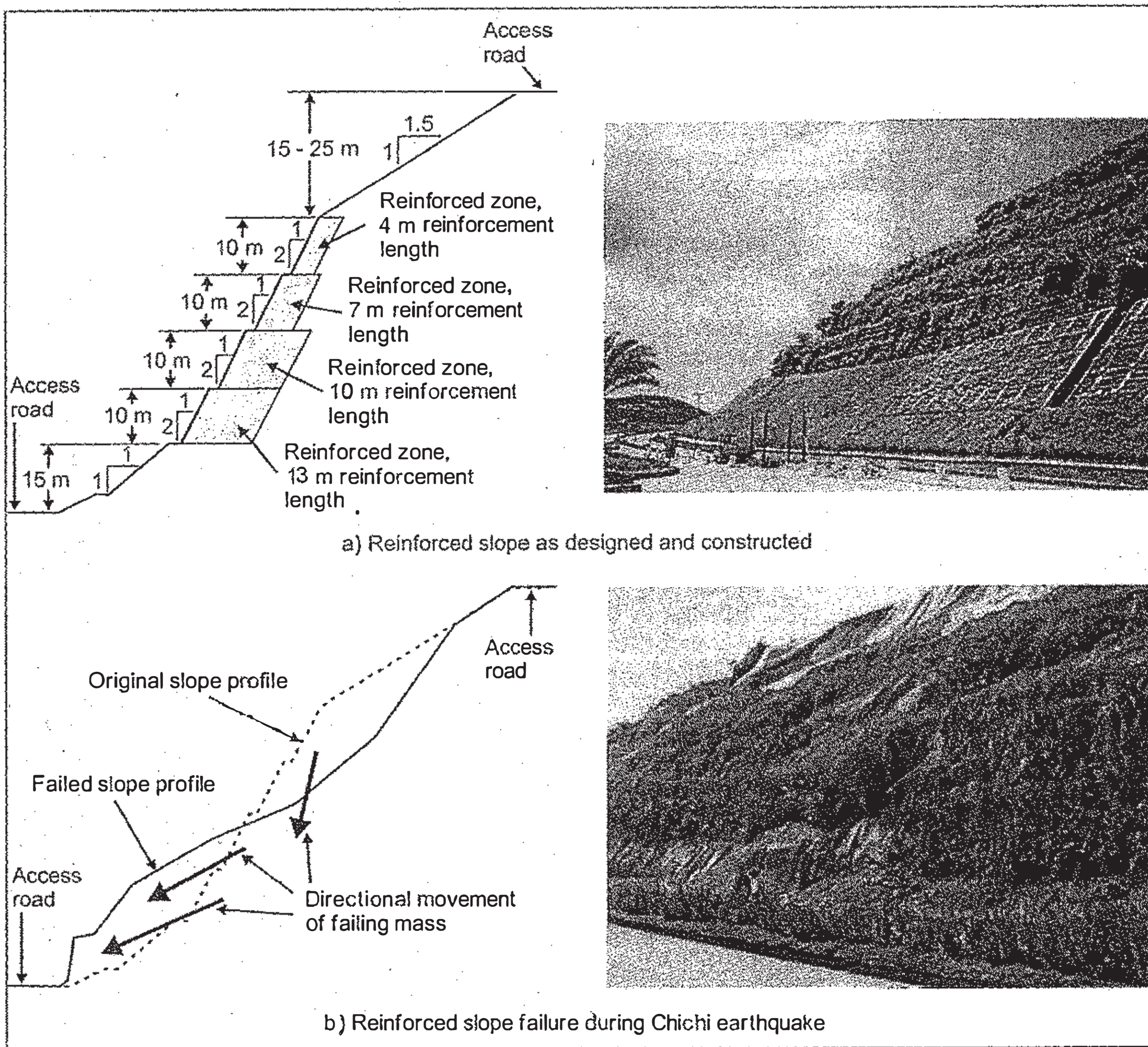

Figure 37. Reinforced fill slope failure, Chi-Nan, Taiwan.

designed and constructed as a competitive alternative to an original reinforced fill slope proposal. The original reinforced slope proposal, which consisted of more-substantial reinforced zones, was more costly than the alternative design, and hence, in preference, the alternative was adopted.

The overall geometry of the slope is shown in Figure 37a with four reinforced tiers, 40 m in height, comprising part of the overall slope height of maximum 80 m. What is unusual about the reinforced slope geometry is that it has a maximum reinforced zone width of only 13 m in the bottom tier and this decreases to 4 m in the top tier. The reinforced fill consisted of the on-site clay-gravel, and geogrids were used as the reinforcement with wrap-around facings and soil bags.

Construction of the Chi-Nan slope began in 1994. Near completion of the construction a fairly large failure occurred in one part of the reinforced slope. This was most likely due to groundwater effects. The failed section of the slope was reconstructed with subsurface drainage being included. The slope was completed in 1996.

On September 21, 1999 a major earthquake of magnitude 7.6 struck the Central West area of Taiwan, the epicentre being near the town of Chichi. Peak ground accelerations as high as 1.0g were recorded near the epicentre. The earthquake was very destructive with up to 2,500 fatalities and much collateral damage.

In the vicinity of the Chi-Nan slope, approximately 20 km to the North East of the earthquake epicentre, severe peak ground accelerations occurred. One strong-motion sensing instrument located near the Chi-Nan slope recorded peak horizontal ground accelerations of 0.60g and peak vertical ground accelerations of 0.28g.

During the strong ground motions the reinforced slope moved outwards and down the toe slope, Figure 37b. Much of the reinforced zones moved enmass with the bottom two tiers sliding horizontally outward and down the toe slope and the upper two tiers slumping into the void left behind. The bottom two tiers came to rest some 15 m down the toe slope at the edge of the access road.

An evaluation of the failed slope has been carried out to determine cause and recommend solutions, Genesis (2000). A back-analysis of the failed slope using limit-equilibrium methods calculated failure at a horizontal acceleration of 0.3g and a vertical acceleration of 0.2g. This assumed that the shear parameters of the clay-gravel reinforced fill were $c' = 20$ kPa and $\phi' = 35°$. These back-analysed accelerations at failure are less than those recorded in the vicinity of the slope thus, slope failure would appear inevitable under the peak accelerations recorded.

Clay-gravel fills are extremely heterogeneous with regard to shear properties due to the presence of the large-sized gravel fractions and the lateritic nature of the undisturbed material. Observed shear properties can range from $c' = 10$ kPa to 120 kPa and $\phi' = 20°$ to 50° depending on the presence of the coarse gravel fraction and the lateritic condition, Woo et al. (1982). The fill shear values used for the back-analysis were determined from insitu large shear box testing, and while the test results varied, the values used were considered an overall best-estimate, site average.

It is interesting to note that it took a severe seismic event to fail the extreme-geometry reinforced slope at Chi-Nan – under static conditions the slope had remained stable for 3 years previous.

Good design and good construction quality are essential for reinforced fill slopes subjected to severe seismic events. The failure of the reinforced slope at Chi-Nan may be compared to the performance of the reinforced slope shown in Figure 38 which was constructed at Nan-Hua in the South West of Taiwan. This slope was also subject to a severe seismic event about one month after the famous Chichi earthquake.

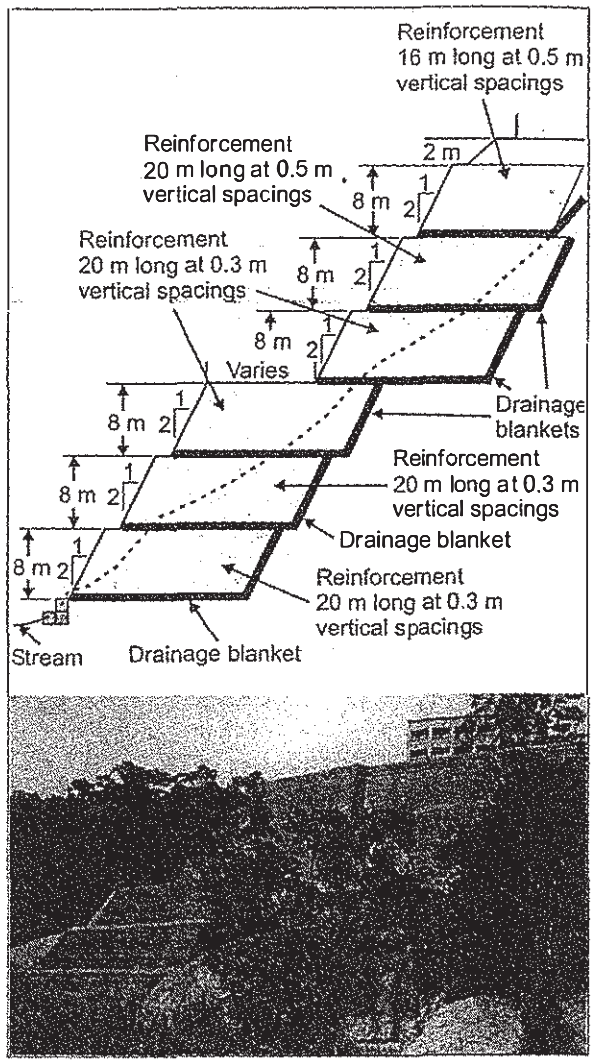

Figure 38. Reinforced fill slope, Nan-Hua, Taiwan.

The reinforced fill slope at Nan-Hua was constructed during 1998 to provide an extra parking and services area for a hillside development. The reinforced slope at its highest point is 50 m and consists of a maximum 6 tiers, each of 10 m in height and 2:1 face slope. The reinforced fill was the local clay-gravel material with design parameters $c' = 0$, $\phi' = 30°$, while the reinforcement used was a polyester geogrid. The slope was designed for seismic loading and the extensive use of drainage was fundamental to the design.

The section shown in Figure 38 depicts a more-conventional reinforced fill geometry compared to the Chi-Nan slope in Figure 37.

The construction quality of the slope was excellent with close attention being paid to compaction of the fill, and the installation of the various drainage measures. The slope facing used was a wrap-around face with soil filled bags, see Figure 31a. For monitoring purposes a series of deformation targets were installed along the slope crest and at selected positions in the parking and services area.

On October 19, 1999 an earthquake with a magnitude of 6.4 struck the South Western area of Taiwan and was centred near the city of Chiayi. While not as destructive as the Chichi earthquake of a month earlier considerable damage and injuries were reported. In the vicinity of Nan-Hua, around 20 km to the South of the earthquake epicentre, a number of natural slope failures occurred, including one slope adjacent to the reinforced slope.

Following the earthquake the reinforced slope showed no signs of damage. This was borne out by an extensive visual inspection. Little deformation occurred at the crest and on top of the slope with results from the deformation targets showing maximum permanent horizontal movements of 2 cm. Some narrow cracks were observed in the asphalt pavement of the car park on top of one part of the slope. On the whole, the performance of the reinforced fill slope was exemplary.

8.10 *Reinstatement of failed slopes using reinforced soil*

Much of Asia is subject to intense rainfall either throughout the year in the equatorial regions or seasonally in the tropic regions. During these periods of intense rainfall many fill slopes, especially embankments, fail. The failures are a result of rainfall either infiltrating the embankment fill directly from the surface or percolating into the fill in the form of groundwater. The water environment within these embankment fills is shown in Figure 39a.

Depending on the procedures adopted during construction the embankment fills can be either poorly compacted or well compacted. Where little compaction has been applied it is relatively easy for water to infiltrate the embankment fill, erode the embankment toe and cause failure of the embankment, Figure 39b. Where compaction of the placed fill has been carried out during construction groundwater flow can cause pore pressures to build up along the interface between the undisturbed soil and the compacted, less-permeable, embankment fill. This pore pressure build up can also cause instability and failure of the embankment fill, Figure 39b.

A number of techniques can be used to reinstate the failed embankment. Reinforced soil is a viable technique because of its use of the local, failed embankment soil for the replacement fill. A typical reinstatement using reinforced soil and drainage is shown in Figure 39c. It is important that the reinforced soil technique be used in conjunction with good drainage for the reinstatement of failed slopes.

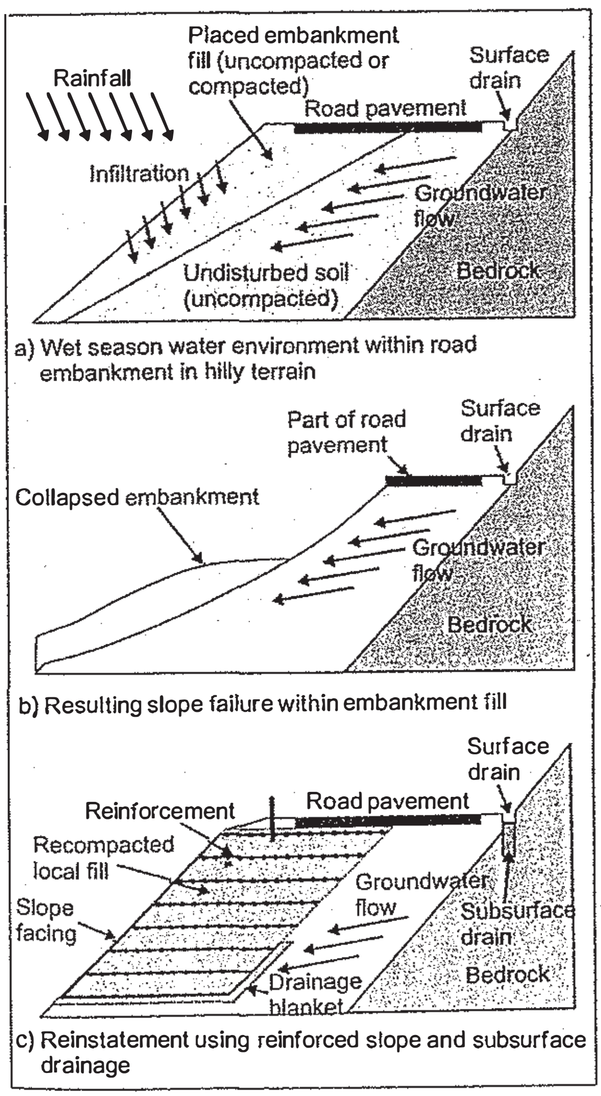

Figure 39. Reinstatement of failed fill slopes following heavy rainfall.

9 CONCLUSIONS

Over the last 30 years reinforced soil has made major advances in Asia both in terms of the number of countries that have used the technique and the range of reinforced soil applications used. Many successful reinforced soil structures have been constructed. However, a number of performance related issues still affect reinforced soil structures as evidenced by some continual problems. It must be emphasized that the number of problem structures is relatively small compared with the total number of reinforced soil structures constructed in Asia.

The performance of reinforced soil structures is dominated by deformation. However, to evaluate the performance of these structures a rational framework incorporating performance and deformation levels needs to be developed. The performance framework needs to account for time as well as deformation levels.

Basal reinforced embankments on soft foundations are essentially a problem of rates. On the one hand there is the rate of loading of the embankment fill and on the other is the rate of gain in shear strength of the soft foundation due to consolidation. The short term misbalance between the two affords the opportunity for basal reinforcement to provide the required additional stability. Changes in the rate of loading and the rate of consolidation can have a significant effect on the quantity of reinforcement required. Critical to performance is a good understanding and knowledge of the behaviour of the soft foundation soil.

Reinforced piled embankments are used extensively where it is necessary to prevent the differential settlement between an embankment fill and a piled-foundation structure. The performance of reinforced piled embankments has been mixed due mainly to short cuts undertaken during the design or construction. Good attention to design methodology is critical for the good performance of these structures. For low-height reinforced piled embankments it is important to recognize that serviceability, with regard to differential surface deformations, is the controlling performance requirement.

Reinforced soil retaining walls have become a standard form of construction in many Asian countries especially in the transportation and mining industries. The problems that have arisen with reinforced soil walls in Asia have inevitably been ones associated with deformation. In the majority of cases this has been due to a lack of appreciation of the impact of the surrounding environment on the performance of the reinforced soil wall.

The major economic advantage of reinforced fill slopes is the ability to use the local insitu soil to construct steep, stable slopes. With much development in Asia moving to the hilly and mountainous hinterland reinforced slopes of considerable height (over 30 m) are common. The problems that have arisen with reinforced slopes in Asia have inevitably been where an unsafe combination of a fine-frictional fill, short reinforcement lengths and groundwater have occurred. In general, problems can be traced back to poor design of the overall dimensions of the reinforced slope and/or poor construction quality. Good and uniform compaction of the fine-frictional fill is essential. Good drainage measures should be mandatory.

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