CONSIDERATIONS IN APPLYING GEOTEXTILES TO COASTAL REVETMENTS

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ABSTRACT

The application of geotextile membranes in breakwater and revetment design raises the issue of the appropriate soil/geotextile and geotextile/geotextile friction angles that can be adopted for stability analysis. A considerable amount of data, much derived from the design of landfills, has been published on this subject. Other data are provided by geotextile manufacturers. Much of the data refer to a variety of woven fabrics, but data exist also for non-woven needle punched geotextiles that are used in coastal engineering structures. This paper reviews the local practice and literature and proposes appropriate values for soil/geotextile and geotextile/geotextile friction angles that may be considered for the preliminary design of coastal revetment structures.

1 INTRODUCTION

The development of modern geotextiles has led to the proliferation of their use in coastal protection revetments. Where embankments of sandy soil require erosion protection with a sloping rock revetment there can be some considerable cost saving in replacing the traditional graded stone filters with a geotextile. Recently, there has been a tendency to replace rock armouring altogether with geotextile sand bags (Figure 1), either as temporary or permanent structures, this being considered by some to be preferable to placing rock on beaches.

Figure 1: Rock and sandbag revetments on Stockton Beach, NSW

The assessment of the stability of a revetment subjected to wave and current action needs to address the stability of the revetment armour units under wave impact (armour stability), the stability of the armour layers on the slope (blanket stability) and the propensity of the entire revetment embankment to slump (global stability). The first is a coastal engineering consideration that, among other things, relates to the permeability of the structure, whereas the latter are geotechnical issues.

The application of geotextiles in coastal revetments has raised issues regarding their permeability as well as the appropriate friction angles that can develop between geotextiles and soils, rock and other geotextiles that should be adopted for stability analysis. Little research has been published on the permeability of geotextiles to wave action. However, a considerable amount of research, derived from the design of landfills, has been published on soil/geotextile and geotextile/geotextile friction angles, the data referring to a variety of fabrics including the non-woven needle-punched (NWNP) geotextiles that are being used now in coastal protection structures.

This paper reviews armour stability and revetment design utilising geotextiles and the literature on soil/geotextile and geotextile/geotextile friction angles. Values for these friction angles considered appropriate for use in the design of coastal protection revetment structures are proposed.

2 TYPICAL COASTAL REVETMENT DESIGNS UTILISING GEOTEXTILES

Typical designs of coastal revetments on sandy soils utilising geotextiles are presented in Figure 2, for a rock revetment designed by the Department of Public Works NSW and Figure 3 for a geotextile sand bag revetment designed by a geotextile supplier. Of particular note is that the revetment slopes in both cases are presented at 1.5H:1V.

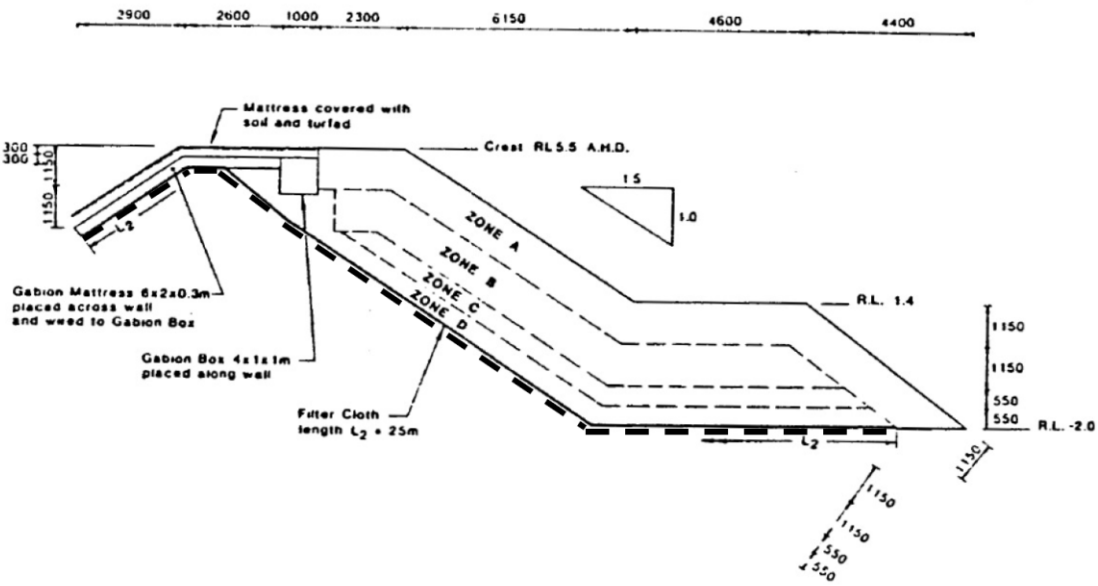


Figure 2: Stockton Seawall Design (source NSW Public Works Department)

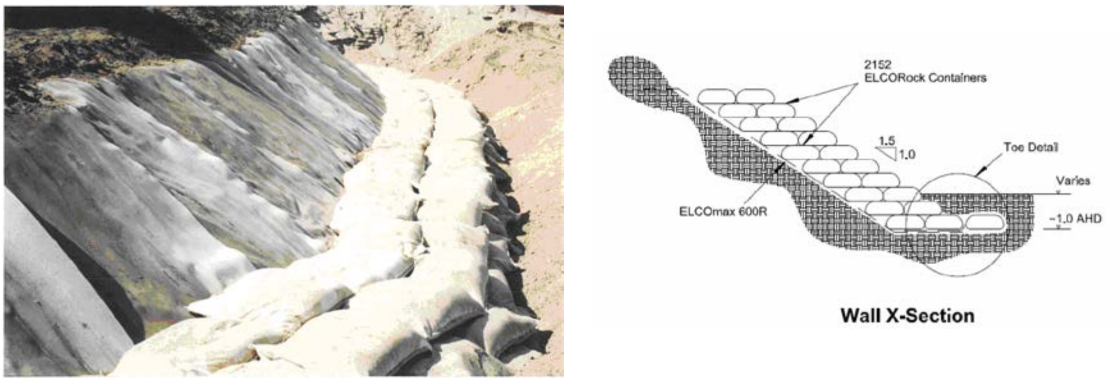


Figure 3: Left: Stockton Surf Club sand bag revetment under construction
Right: Typical geotextile sand bag revetment cross-section
(*source Geofabrics Australasia ELCO Solutions*)

3 GEOTEXTILE PROPERTIES

3.1 PERMEABILITY

Geotextiles have been developed to provide a separating layer between the subsoil and rock armouring that prevents the egress of the soil, does not inhibit the flow of groundwater and is robust in construction. The NWNP geotextiles used in coastal revetments are manufactured using fibres that are bound to each other by mechanical needling. This optimises the balance between hydraulic and mechanical properties for:

- high permeability with fine filtration
- high puncture resistance with high strains to failure
- good cushioning ability and impact resistance

- high in-plane flow capacity.

In many instances it is recommended that armour be placed directly on top of the geotextile so there is no need for an intermediate bedding layer of stone.

The geotextile grade is selected, among other things, so that its permeability is at least one order of magnitude greater than the permeability of the subsoil. Typically, geotextiles have a coefficient of permeability in the order of 10 m/s. Pore size is in the order of 100 μm and, under 100 mm head, flow rates are in the order of 100 l/m^2/s.

3.2 GEOTEXTILE/SOIL FRICTION

Research results presented on soil/geotextile friction properties are presented variously as friction angles (φ_{sg}) or as a Coefficient of Interaction, *CI*, where

$$CI = \tan \varphi_{sg} \ / \tan \varphi_s \qquad (1)$$

where φ_s is the soil friction angle.

The statistical approach adopted herein treats each datum point as though it represented the average of the shear strength for a site (i.e. site sand and geotextile) and, thus, the dispersion of the data represents situations on different unknown sites. It is considered that, for the data, this is approximately true and that the estimates so obtained can be applied to preliminary design on other sites where there are no site specific data.

Exxon (1992), a former manufacturer of NWNP geotextiles, recommended a *CI* value of 0.7–0.8. This value would result in φ_{sg} = 24°– 28° for φ_s = 32°–34° (typical values for coastal sands).

Tencate Geosynthetics Asia (www.tencate.com) reported test results from a 500 mm × 500 mm direct shear box apparatus with their product Polyfelt®TS on sand with φ_s = 40° and 41° under confining stresses of 10 kPa to 60 kPa that yielded φ_{sg} = 29° and 32° respectively, giving *CI* values of 0.66 and 0.72. It is noted that the φ_s values are high for coastal sands and probably more relevant to a manufactured sand (i.e. crushed rock).

Geofabrics Australasia (Hornsey/Nielsen personal communication) has reported the results of a single 300 mm × 300 mm direct shear box test carried out on their product Terrafix 1200R with sand by Naue Fasertechnik that yielded a result for the residual strength of φ_{sg} = 30.9º. No data were provided on the internal frictional strength of the sand used in the test.

Dixon *et al*. (2006) analysed a large dataset from laboratory tests on interfacial shear stress versus confining stress that was gleaned from the available published literature as well as their own unpublished research. For the peak strength data for NWNP geotextiles, a line of best fit gave a friction angle of φ_{mean} = 35.0° with an apparent cohesion of 3.6 kPa. The apparent cohesion is an artefact of the line of best fit procedure. If it is assumed that the material properties are frictional without cohesion and the line of best fit is forced through the origin of the data set, φ_{mean} = 36.4°. At a confining stress (σ_n) of 50 kPa (as is appropriate for the shallow conditions pertaining to revetments), for the standard deviation (SD) of 0.155 σ_n (as given in Dixon *et al*., 2006) and adopting a characteristic value being the mean minus 1.5 SD (ensuring about 90% of the data lie above the value) results in φ_{sg} = 26.8°.

The data for the residual shear strength of NWNP geotextile/sand friction angle under low confining stress are plotted in Figure 4. For the entire data set (not shown here), a line of best fit gave a friction angle of φ_{mean} = 34.2° with an apparent cohesion of 4.2 kPa. If the line of best fit is forced through the origin of the data set, φ_{mean} = 35.8°. For a standard deviation (SD) of 0.136 σ_n at a confining stress of 50 kPa, adopting a characteristic value (being the mean minus 1.5 SD) results in φ_{sg} = 27.3°. It is noted here that quite a few data points fall below the line of best fit, which probably led Dixon *et al*. (2006) to suggest adopting a characteristic value being the mean minus 3 SD where there is a paucity of data.

Koerner and Narejo (2005) reported a summary of collected and in-house data on interface shear strength for a number of interfaces including NWNP geotextiles to granular soils. The data were from a large number of projects and soils (290 individual tests for peak strength, 117 for residual strength) with a wide confining stress test range of 5 kPa to 660 kPa. Taking account of the low confining stress peak strength data reported in Koerner and Narejo (2005), as shown in Figure 5, to adopt a soil/geotextile friction angle for which most of the low confining stress data exceeded would result in a friction angle of φ_{sg} = 23°.

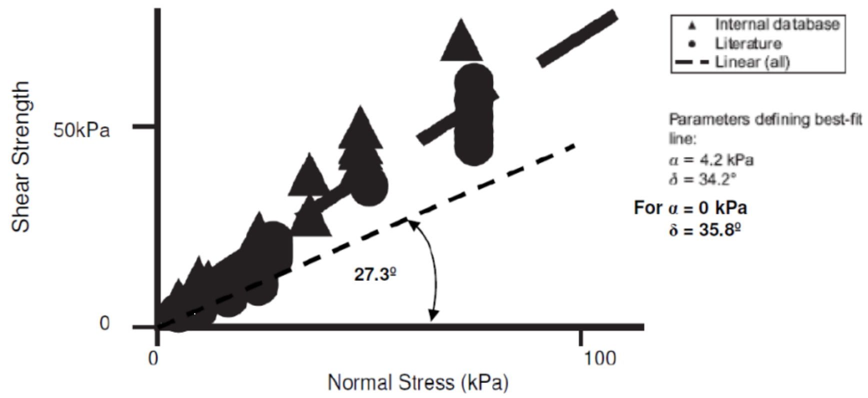


Figure 4: Residual Shear Strength versus Normal Stress for low confining stress data (modified after Dixon *et al*., 2006)

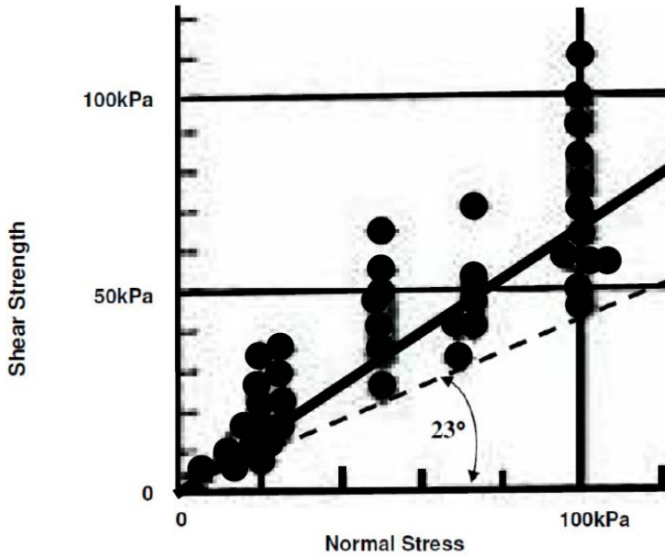


Figure 5: Peak interfacial shear strength versus normal stress for low confining stress data (modified from Koerner & Narejo 2005)

Martin *et a*l. (1984) presented one test result of interfacial friction angle undertaken on NWNP geotextile with Ottawa sand, being 26°, and two results for concrete sand, being 26°–30°. The sand friction angles given for these materials was 28° and 30° respectively, which result in high *CI* values of 0.9 to 1.0. These sand friction angles appear low when compared with 38° and 36° (respectively) as reported in Williams and Houlihan (1987). It is noted here that for Ottawa Sand, friction angles commonly reported in the literature range from 28° to 35° depending on relative density (Holtz and Kovacs, 1981).

Williams and Houlihan (1987) advocated the use of a large shear box (305 mm × 305 mm), for which the *CI* value taken from the published literature was reported to be around 0.9 for NWNP geotextiles with clean sands.

Tan *et al*. (1998) advocated the use of a large torsional ring shear apparatus to evaluate the residual shear strength of the soil-geotextile interface using NWNP geotextiles and uniform medium sand. The study results are summarised in Figure 6 and indicated that the friction angle for the residual shear strength was between 24º and 27º. The friction angle of the sand was not given but the material was described as medium sand at a relative density of around 0.55. Comparisons with direct shear tests indicated that the latter gave identical friction angles at small displacements (less than 3 mm) but higher friction angles than the ring shear apparatus at larger strains, which was attributed to a deficiency in the direct shear box apparatus to measure shear stress beyond displacements of 15 mm.

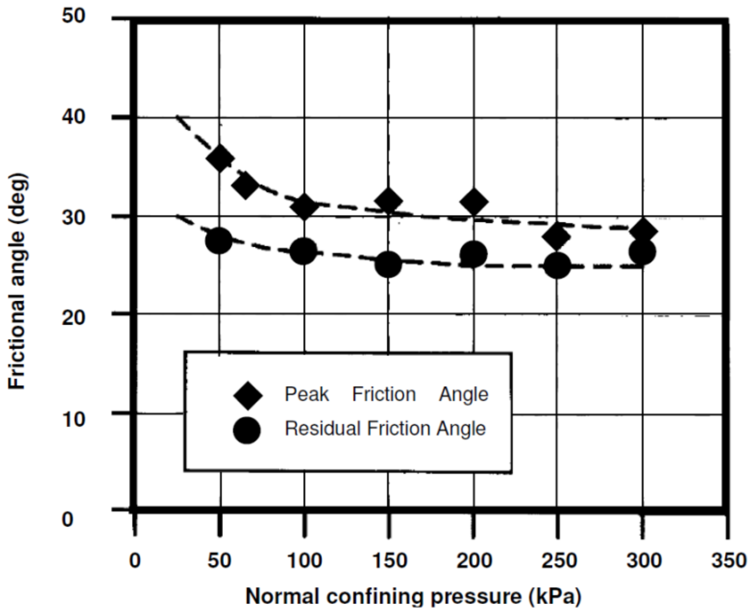


Figure 6: Apparent friction angle of sand-geotextile interface versus normal confining pressure (after Tan *et al*., 1998)

Table 1: Summary of results and interpretations for residual φ_{sg} and *CI* from literature review

Source	φ_s	*CI*	φ_{sg}	Comment
Exxon (1992)	NA[2]	0.7–0.8	NA	Recommended in manual
Geofabrics Elco	NA	NA	31°	Single test φ_s not given
Tencate Polyfelt	40°– 41°	0.7	29°–32°	High φ_s
Dixon *et al*. (2006)	NA	NA	27°(1)	Large data set φ_{sg} calculated at 50 kPa confining stress. φ_{sg} calculated for shear stress minus 1.5 SD φ_s not given
Koerner and Narejo (2005)	NA	NA	23°(1)	Large data set φ_s not given
Martin et al. (1984)	28° Ottawa Sand(3) 30° Concrete Sand(4)	0.9 0.8 – 1.0	26° 26° – 30°	Comparatively low values for φ_s, at the low confining stress tested
Williams and Houlihan (1987)	38° Ottawa Sand(5) 36° Concrete Sand(6)	0.6–0.7 0.7–0.9	25° (0.8kPa) to 28° (1.4kPa) 27° (0.8kPa) to 34° (1.2kPa)	Apparent cohesion in brackets *CI* calculated at 50 kPa confining stress
Tan *et al*. (1998)	NA	NA	24°–27°	φ_s not given

(1) Values for which 90% of data exceed φ_{sg}
(2) NA – Not Available
(3) d_{10} = 0.42 mm ; Coefficient of Uniformity 1.9; rounded
(4) d_{10} = 0.20 mm ; Coefficient of Uniformity 2.6; angular
(5) d = 0.6 – 0.8 mm
(6) Limestone sand

The results and interpretations from the literature review are summarised in Table 1 and indicated that, in the absence of site specific field data and given the interpretative comments, an interfacial friction angle between NWNP geotextiles and coarse sand of φ_{sg} = 25° would ensure appropriate conservatism in concept design (90% exceedance). Alternatively, if the friction angle of a coarse sand was known from testing, adopting a Coefficient of Interaction of CI = 0.7 would ensure appropriate conservatism in concept design. Site specific data are likely to allow for adopting higher values.

Dixon *et al*. (2006) stated that "design based wholly on literature values should not be attempted" . . . and . . . "it is proposed that these summaries of test data can be used to supplement site specific test results in order to select appropriate mean and standard deviations for interface shear strength" . . . and . . . "In some cases, literature values are being used in lieu of site specific test results, and this is considered [to] be unacceptable and likely to lead to unreliable designs". This was supported by Williams and Houlihan (1987) stating that "The friction analyses should be performed using the site soil, placed and compacted in a manner which simulates field conditions". Thus the recommendations in the previous paragraphs are considered appropriate for concept or preliminary design purposes only.

3.3 GEOTEXTILE/GEOTEXTILE FRICTION

Tencate Geosynthetics Asia (www.tencate.com) reported test results from a 100 mm × 100 mm and a 500 mm × 500 mm direct shear box apparatus for geotextile/geotextile friction angle (φ_{gg}) with their NWNP product Polyfelt®TS that yielded φ_{gg} = 20° and 18° (respectively).

Geofabrics Australasia (Hornsey/Nielsen personal communication) has reported the results of a single 300 mm × 300 mm direct shear box test carried out on NWNP Terrafix 1200R (Elcomax®1200R) by Naue Fasertechnik that yielded a result for the residual strength of φ_{gg} = 20º.

Oumeraci and Recio (2010) gave a range of φ_{gg} = 20º – 26° for NWNP geotextiles.

In lieu of site specific testing, adopting φ_{gg} = 18º would allow appropriate conservatism for concept design.

4 REVETMENT STABILITY

4.1 ARMOUR STABILITY

The stability of armouring varies considerably with the permeability of the core; that is, the permeability of the material below the armour and its underlayer. The less permeable the core the greater the amount of wave energy that is reflected off the structure and back onto the armouring. If a geotextile is used as a filter layer beneath the armour layers then the core is to be considered impermeable to wave action (CIRIA 2007, p 566). In such cases, for rock armour the mass of armourstone would need to be around four times larger than that required for a permeable core (CIRIA 2007, p 566). It is noted that all of Hudson's testing (Hudson, 1959) for rock armour was undertaken on models with permeable cores (CIRIA 2007, p 566) and the stability factors (K_D) given for Hudson's equation (see CERC, 1984) need to be reduced significantly if a geotextile is to be placed beneath the armour layers (CIRIA 2007, p 566). If the van der Meer (1992) equations are to be used for structural design, then the requisite armourstone mass would be some 3 times greater than that required if geotextile was not used.

Other issues relating to utilising a geotextile immediately beneath rock armour layers are its durability under cyclic wave and tidal loading and its direct exposure to the elements through the interstices of the rock armouring. Oumeraci and Recio (2010) suggested a lifetime in the order of 20 to 25 years if vandalism and damage during construction can be avoided.

4.2 BLANKET STABILITY

Geotextiles may introduce a shear surface detrimental to the stability of the armour layer (Oumeraci & Recio, 2010). The stability of the armour blanket against sliding on the face of the revetment relies on the interfacial friction between the armour layer and the retained soil. If a geotextile is to be used between the armour layers and the soil, consideration needs to be given to both the interfacial friction between the armouring and the geotextile as well as the interfacial friction between the geotextile and the retained soil.

Factors of safety (FoS) against blanket sliding failure of around 1.5 commonly are accepted for these cases. However, larger values may be considered, given the dynamic nature of the applied loadings.

If the retained soil embankment comprised sand, the internal friction angle for the sand of φ_s = 35º commonly is adopted although lower values are often encountered. For CI = 0.7, the interfacial friction angle between the sand and the geotextile would be φ_{sg} = 26º, a value around that found by independent researchers in the literature review. For a geotextile interface to be stable with a FoS = 1.5, the slope would need to be no steeper than

3H:1V (see Box 1) unless the design incorporated intentional waviness (large scale roughness elements) and construction paid particular attention to this.

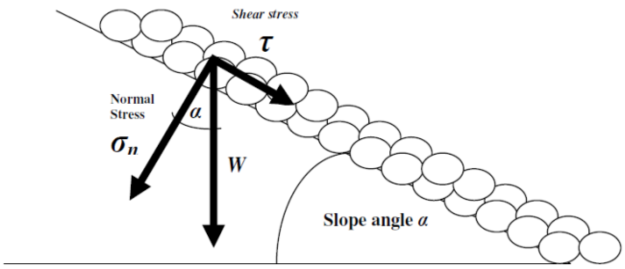
If the armour units comprised geotextile sandbags, for an interface friction angle between the sandbags and the geotextile underlayer of φ_{gg}, of around 20°, for a FoS of 1.5 against sliding the revetment slope would need to be flatter than 4H:1V (see Box 1).

4.3 GLOBAL STABILITY

The use of sandbag type elements in revetment structures introduces many complications to stability analysis. Most limit equilibrium programs, such as SLOPE/W and SLIDE, can deal with these complications provided they are used properly. To illustrate some of the problems with the analysis and design of these structures, several typical cases have been analysed. These cases represent the situation that is likely to arise if there is no project specific testing undertaken on the retained sand and geotextile interfaces. The analysis has assumed:

- A batter of 1.5H:1V
- The retained sand is a typical cohesionless loose dune sand with φ_s of 32º and a unit weight, γ, of 16 kN/m^3
- The bags have a geotextile/geotextile shear strength equivalent to a φ_{gg} of 18º and an addition to the friction angle of 5º resulting from the large scale irregularities of the interfaces, resulting in φ_{gg} of 23º, and a unit weight, γ, of 18 kN/m^3
- There is no geotextile layer as an interface between the sandbags and retained soil and that the sandbags are arranged such that shear on the interface is through sand (i.e., that the interface is very rough with the sandbags stepped)
- A target FoS of 1.5 with the water table assumed below the failure surface

Box 1: Stability analysis of a sandbag revetment against sliding



$$\tau = W \sin\alpha$$

$$\sigma_n = W \cos\alpha$$

$$\tau/\sigma_n = \tan\alpha$$

For geotextile bags sitting straight on a sand slope (α) with a FoS = 1.5:

$$\tau = 1.5\ \sigma_n \tan\alpha$$

but

$$\tau = \sigma_n \tan\varphi_{gs}$$

$$\tan \alpha = \tan \varphi_{gs} / 1.5$$

Therefore, the safe slope angle is:

$$\alpha = \arctan(\tan \varphi_{gs}/1.5)$$

If the sandbags sit on a geotextile underlayer with a FoS = 1.5:

$$\tau = \sigma_n \tan\varphi_{gg}$$

and the safe slope angle is:

$$\alpha = \arctan(\tan \varphi_{gg}/1.5)$$

For an armour layer 3 m thick (e.g., Figure 7), the following results were obtained:

- For an optimised non circular failure surface, the maximum height for the target FoS was 2.8 m.
- If the bed-courses are modelled with circular failure surfaces, restrained to be horizontal through the armour, the maximum height for the target FoS is 5 m. Such slip circle analyses are non-conservative and do not identify the critical failure surface, which is not circular for frictional materials but, in the authors' experience, are commonly adopted.
- If the sand is modelled with a curved failure envelope equivalent to φ_s of 32º over a normal stress range of 30 to 250 kPa, the maximum height for the target FoS is 4 m. It is the authors' experience that often a slight curvature of the failure envelope can explain the stability of low height slopes without recourse to "apparent" cohesions in free-running sand.
- Tilting the bed-courses at 5 to 10º degrees into the slope makes only a minor difference to the computed FoS and maximum heights provided above.

For an armour layer 1.2 m thick, the maximum height for the target FoS (1.5) was less than 2 m, and approximately 2.5 m with the curved strength envelope for the retained sand.

It was noted that with the base case described above, the FoS for a 9 m high batter was approximately 1.0 for both the 3 m and 1.2 m thick sand bag armour layers. This indicated that, even though many such batters may have been constructed, it is not necessarily the case that each, or any, had an acceptable FoS.

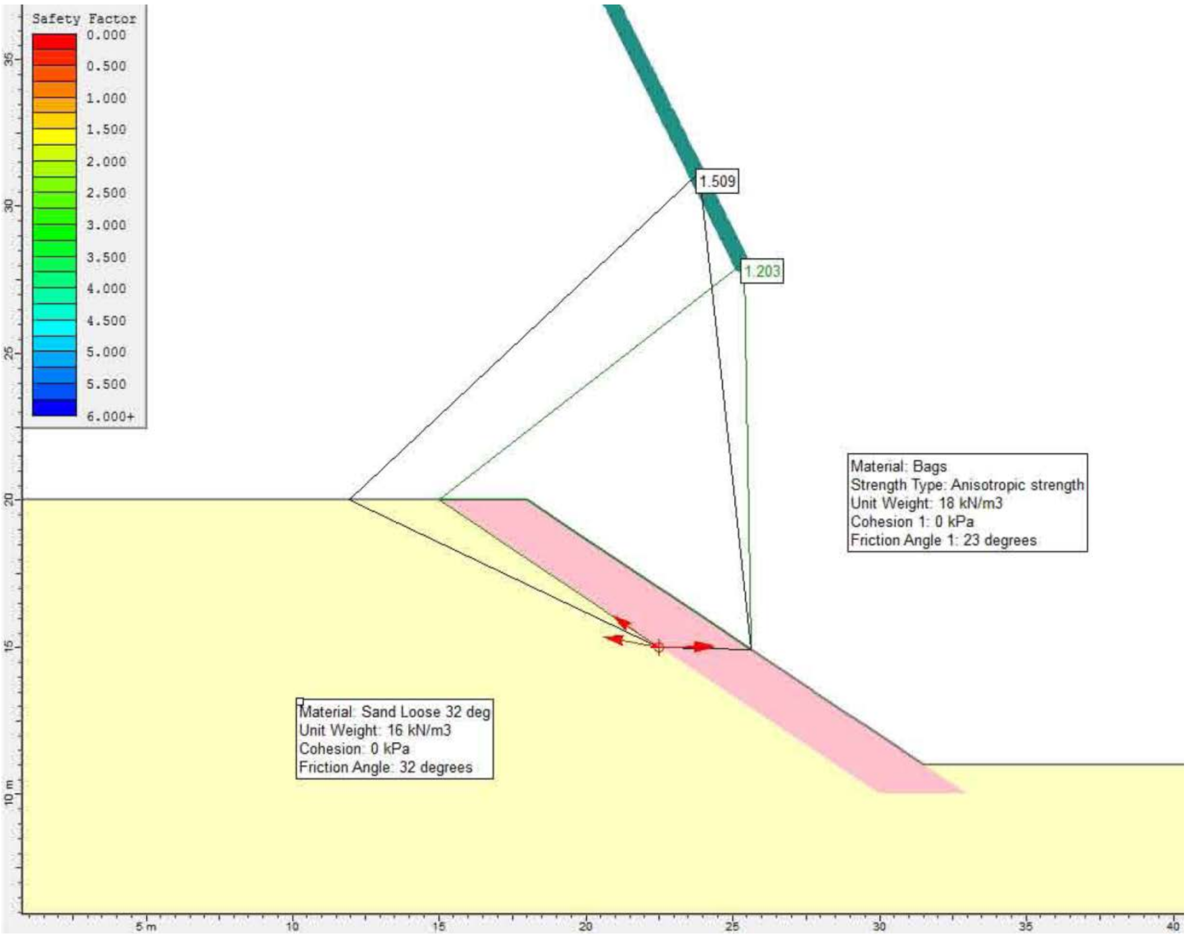


Figure 7: Results of a SLIDE analysis for a 3 m thick sandbag revetment with no geotextile underlayer

5 DISCUSSION

It is common to see revetment armouring on sandy soils constructed to slopes as steep as 1.5H:1V (e.g. Figures 2 & 3). Such steep slopes are likely to have a global FoS < 1.5 for any height above 3 m even with a thick armour layer. This could be unacceptable for a public space. Such designs are unlikely to have taken sufficient account of the global stability of the embankment they have been designed to protect. If such steep slopes are required then they would require site specific design parameters and detailed stability analysis and even then may not be feasible.

The Water Research Laboratory of the University of NSW has recommended that ELCOROCK® revetments, which comprise NWNP geotextile bags laid on a NWNP geotextile underlayer (Figure 3), be built to a 1.5H:1V structure slope with a double-layer "stretcher bond" arrangement (Coghlan *et al*., 2009). Such a design does not appear to have taken into consideration the global stability or the published information on interfacial shear strengths.

Oumeraci and Recio (2010) presented an example of sandbag armour units having unravelled on a dune revetment (Figure 8). A similar example can be found at Byron Bay, NSW (Figure 9). These failures may be attributed to a variety of causes, of which the low friction properties of the geotextile interfaces may be a contributor.

Oumeraci and Recio (2010) presented also examples of stable geotextile-reinforced revetments designed as gravity structures (Figure 10). With this design the low friction angle associated with geotextiles is recognised by the relatively large cross-shore width that has been designed for the potential failure planes. A gravity wall constructed with sandbags, as modelled in Section 4.3, has been analysed. The geometry of the wall is 1.2 m thick at the crest, a front batter of 1.5H:1V and a varying rear batter. Typical arrangement is shown on Figure 11. For a target FoS of 1.5, the maximum heights obtained are given in Table 2.

Figure 8: Sandbag pull-out on a steep dune revetment (Oumeraci & Recio 2010)

Figure 9: Sandbag revetment at Belongil Spit, Byron Bay, NSW 8th June 2011
(*photo courtesy Manly Hydraulics Laboratory*)

Table 2: Maximum heights of gravity sandbag retaining walls with facing slope 1.5H:1V

Rear batter	**1H:1V**	**0.75H:1V**	**0.5H:1V**	**0.25H:1V**
Maximum height	<2 m	2.5 m	7 m	9 m

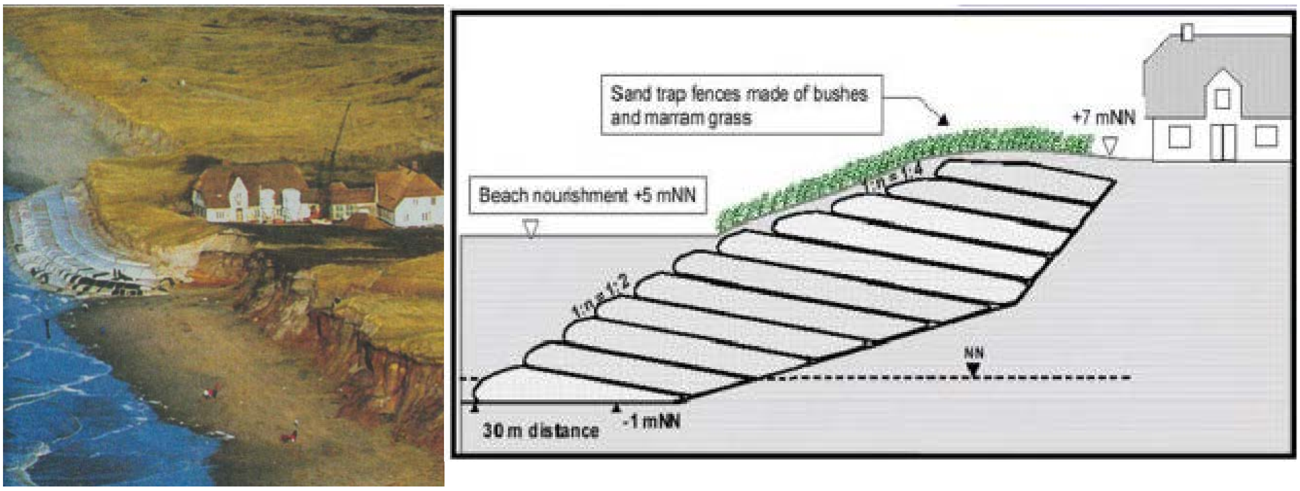


Figure 10: Geotextile-reinforced gravity revetment on the Island of Sylt, Germany (Oumeraci & Recio 2010)

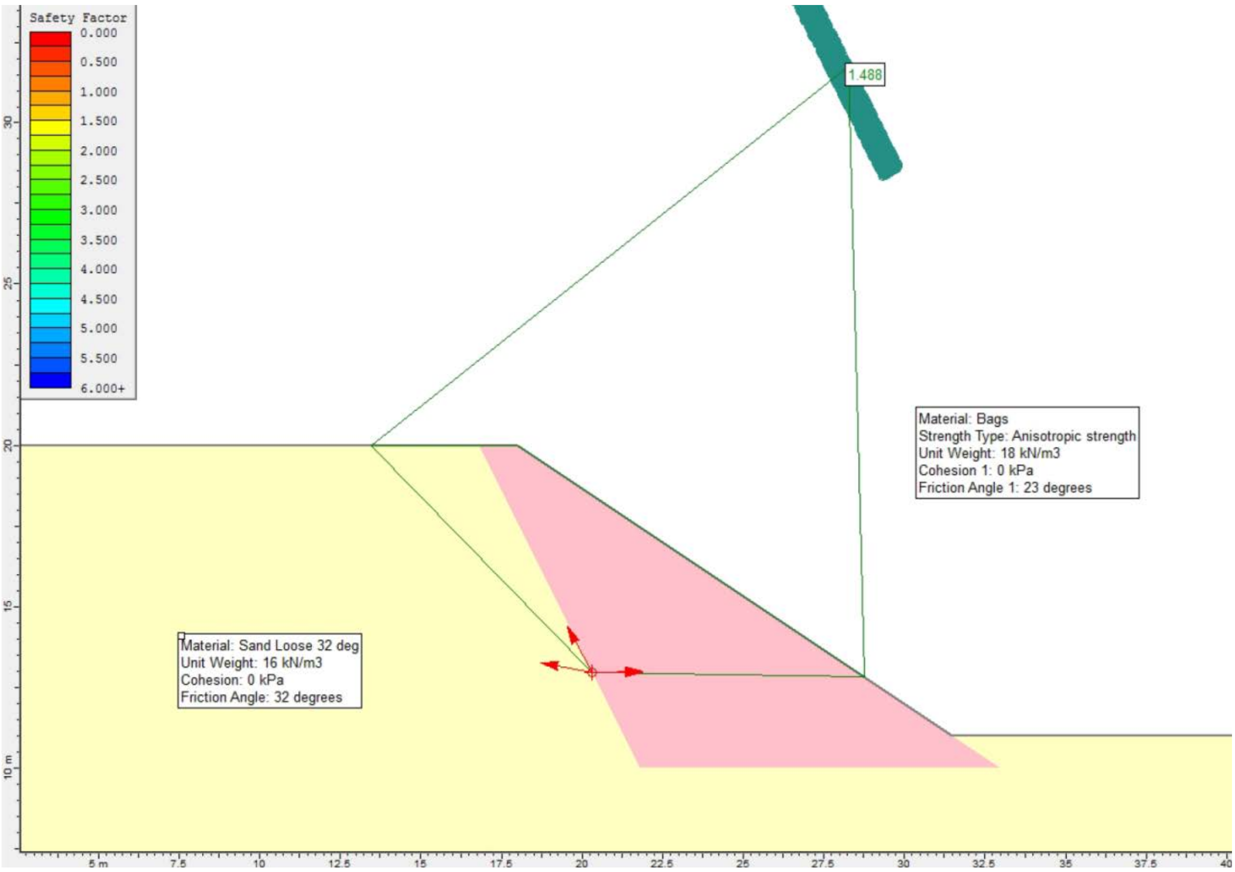


Figure 11: Results of a SLIDE analysis for an 8 m high sandbag gravity retaining wall.

6 CONCLUSIONS

A literature review has been undertaken to assess what may be the appropriate friction parameters to be adopted for utilising non-woven needle-punched geotextiles in coastal revetments. Generally, the literature appeared to be consistent and, for preliminary text-book designs of revetments using non-woven needle-punched geotextiles, the following parameters are recommended for sand/geotextile (φ_{sg}) and geotextile/geotextile (φ_{gg}) friction angles (respectively):

$$\varphi_{sg} = 25°$$
$$\varphi_{gg} = 18°$$

A corresponding value of the Coefficient of Interaction (*CI*) for φ_{sg} would be 0.7. These values are considered to be conservative and are recommended for preliminary design only. It is possible that a larger scale roughness that may develop between sandbags might account for increased friction on the geotextile-geotextile interface (φ_{gg}) and an additional 5° has been assumed in the analyses undertaken herein (φ_{gg} = 25° between sandbags). On a sandbag-sand interface, friction up to the sand strength could be developed should the bags be stepped roughly. However, no research results on these aspects were found.

For utilising geotextile underneath a rock armoured revetment on a sandy soil slope, in lieu of any site specific data and notwithstanding the influence of any larger scale roughness elements, preliminary geotechnical analysis has indicated that geotextile/sand interface slopes would not have an adequate factor of safety (FoS = 1.5) against slip unless they were flatter than 3H:1V. If geotextile is to be used as a separator underneath rock armouring, rock sizing must take into consideration the impermeability of the geotextile to hydrodynamic wave impact. In such cases the rock armour mass is likely to be some four times greater than that which otherwise would be used.

For utilising geotextile underneath a geotextile sand bag armoured revetment, the geotextile/geotextile interface slopes would not have an adequate factor of safety against slip unless they were flatter than 4H:1V. However, preliminary geotechnical analyses have indicated that geotextile-reinforced gravity structures could provide stable revetments in sand (FoS = 1.5) at a facing slope of 1.5H:1V provided that the cross-shore width of the structure was of the same dimension as the retained height, notwithstanding any requirement for wave action.

It is reiterated that text book designs should not be attempted for anything else but concept design and it is recommended that final designs be based on site specific data and rigorous geotechnical analyses. Project specific testing, careful design, rigorous analysis and detailed construction methods and supervision may allow safe batters to be steeper than those indicated herein.

7 ACKNOWLEDGEMENTS

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