SOIL-GEOSYNTHETIC INTERACTION: MODELLING AND ANALYSIS
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Abstract: Interaction between soils and geosynthetics is of utmost importance in applications of these materials as reinforcement in geotechnical engineering. That is also true for some applications of geosynthetics in environmental protection works. The mechanisms of soil-geosynthetic interaction can be very complex, depending on the type and properties of the geosynthetic and the soil. This paper presents and discusses some experimental and theoretical methods for the study and evaluation of interaction between soils and geosynthetics, with particular reference to the applications of these materials in soil reinforcement. The main advantages and limitations of some traditional experimental and theoretical methods for the study of soil-geosynthetics interaction are presented and new applications of these methods are also addressed. The need for improvements in experimental and theoretical techniques for a better understanding of soil-geosynthetic interaction is highlighted.

Keywords: Geosynthetics, Soil-Geosynthetic Interaction, Testing, Analysis.

INTRODUCTION

Soil-reinforcement interaction is of utmost importance for the design and performance of reinforced soil structures and this interaction can be very complex, depending on the nature and properties of the reinforcement. Some geosynthetics used as reinforcing materials can add to the complexity of the problem because of their geometrical characteristic or time and strain rate dependent behaviour. Therefore, different types of tests and theoretical analyses have been developed during the last decades in order to improve the understanding of soil-reinforcement interaction, with particular reference to the use of geosynthetic materials.

Some of the test devices developed has become classical and Figure 1 (Palmeira & Milligan 1989a, Mendes et al. 2007) to some extent helps to understand the reasons for that. This figure shows some possible failure or deformation mechanisms of a reinforced soil wall depending on the region and loading conditions considered. In region A (Fig. 1) sliding of the soil mass on the reinforcement surface can occur, so direct shear tests can be employed to quantify soil-reinforcement bond under these circumstances. In region B, soil and reinforcement can deform laterally, so a plane strain test similar to the in-soil tensile test can be used in this case. Region C shows a situation where soil and reinforcement are sheared, so the direct shear test with the reinforcement inclined to the shear plane can be employed. In region D the reinforcement is being pulled-out, so pull-out tests would be applicable. It should be pointed out that all these test types have limitations in simulating the actual conditions found in a reinforced soil structure.

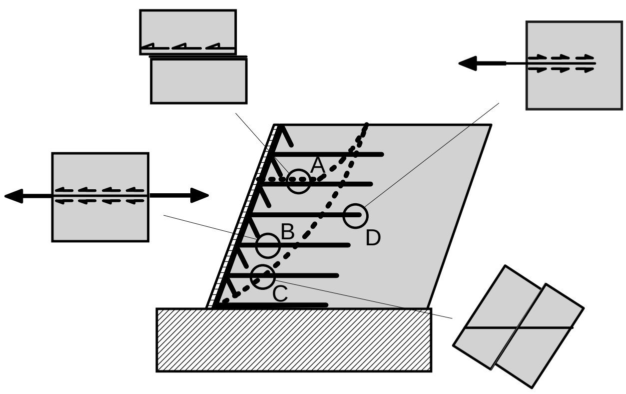


Figure 1. Interaction mechanisms in a geosynthetic reinforced soil wall (modified from Palmeira and Milligan 1989a).

The aim of this paper is to investigate interaction between soils and geosynthetics. The idea behind it is to discuss experimental and numerical modelling tools for the understanding and evaluation of such interaction, which is relevant to most of the applications of these materials. In the following sections, the characteristics and limitations of some tests like those depicted in Figure 1 are presented and discussed, as well as their potentials for the study of other relevant aspects of soil-geosynthetic interaction, besides the adherence between these materials.

DIRECT SHEAR TESTS

Different apparatus and boundary conditions can be found in the literature for the direct shear test apparatus. Figures 2(a) to (d) present some of these conditions. The bottom halve of the box can be occupied by the same soil as the top, a different soil or a rigid block. Usually the main differences among test arrangements are related to the way the geosynthetic is fixed and to the procedure adopted to apply the normal stress to the soil sample top. This stress can be applied by a rigid and free top plate, a rigid top plate not allowed to rotate, a top plate fixed to the top halve of the cell or a flexible pressurised bag. In spite of some advantages (Jewell 1981), the use of a top plate fixed to the box top halve is limited to tests on dilative samples. Pressurised bags are more practical for large scale devices and guarantee a uniform normal stress distribution along the sample surface. One should expect that different boundary conditions are likely to produce some differences in test results.

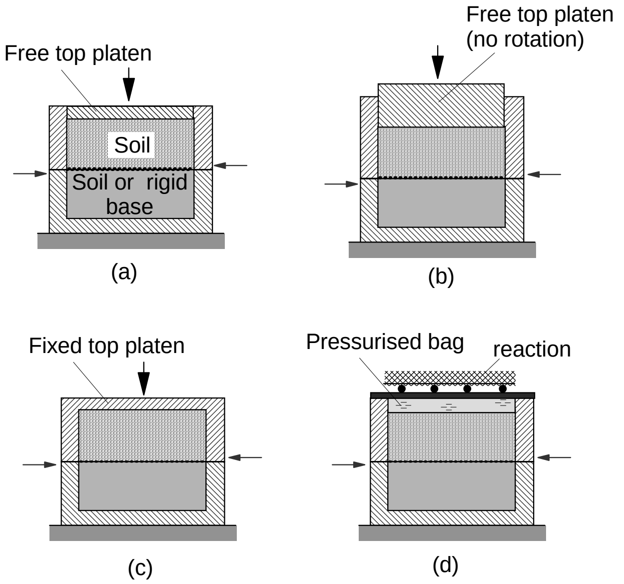
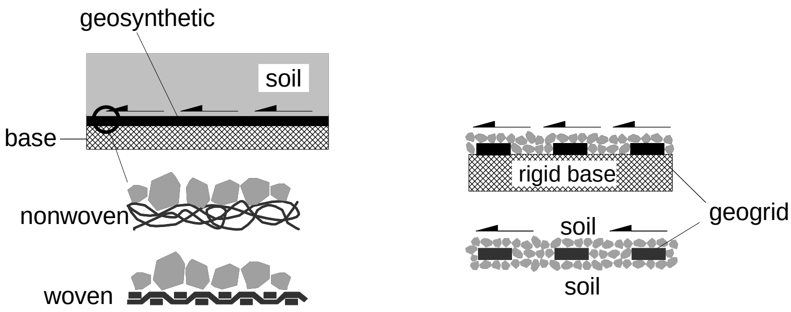


Figure 2. Typical boundary conditions in direct shear test devices.

In direct shear tests with the reinforcement at box mid-height one has to take into account the type of reinforcement which is being tested for a proper analysis of the test results. In the case of geotextiles, a rather uniform shear mechanism will develop, and some interlocking between soil particles and geotextile fibres may take place (Fig. 3a), depending on the particle dimensions and surface characteristics of the geotextile. In tests on geogrids one can have a test arrangement where a rigid base or soil is placed underneath the grid (Fig. 3b). In such cases, friction will be shared by the soil particles in between the transverse members and the solid surface of the geogrid. Jewell et al. (1984) have derived expressions to evaluate bond between soil and geogrid under such conditions.



(a) Continuous geosynthetics. (b) Geogrids.

Figure 3. Soil-geosynthetic interaction in direct shear tests with the reinforcement at the box mid-height.

One aspect related to the result of a direct shear test on a horizontal nonwoven geotextile at the sample central plane is that the result obtained is likely to be influenced by the geotextile distortion. Figure 4 shows direct shear test results with the traditional arrangement of a nonwoven geotextile at the box mid-height (Tupa 1994). Tests were performed on the soil alone (a fine sand) and on the sand-geotextile interface. An additional test with the geotextile fixed to two rigid blocks was also carried out subjecting the geotextile to simple shear conditions. The results obtained show that the initial shear displacements of the soil-geotextile interface were those caused by the distortion of the geotextile. After some level of geotextile fibre stretching have occurred, relative movement between soil particles and geotextile took place. This behaviour is relevant to numerical analysis of reinforced soil structures, because numerical solutions require interface shear stiffness as one of the input parameters. Numerical analyses conducted by Hatami and Bathurst (2005) have shown that the soil-reinforcement interface stiffness can have a significant influence on the lateral displacements of a reinforced wall face. So, due care must be taken when choosing the value of the soil-nonwoven geotextile interface shear stiffness, as it may be dependent on the geotextile shear stiffness. Under field conditions this influence is likely to depend on the construction characteristics and on the level of impregnation of the geotextile by soil particles, as schematically shown in Figure 5. If the geotextile is heavily impregnated, the mobility of the fibres will be restricted and distortion under simple shear conditions is likely to be small.

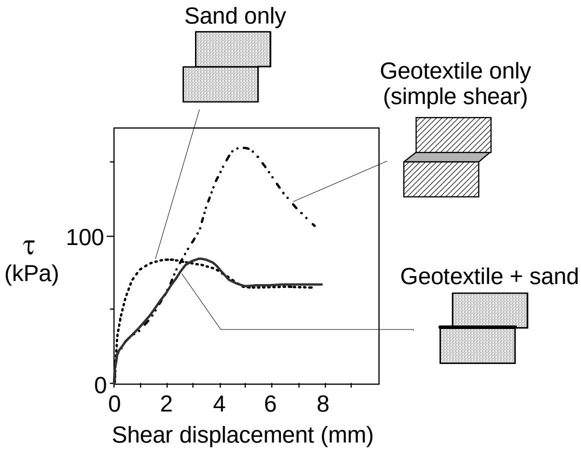


Figure 4. Influence of nonwoven geotextile distortion on the results of direct shear tests.

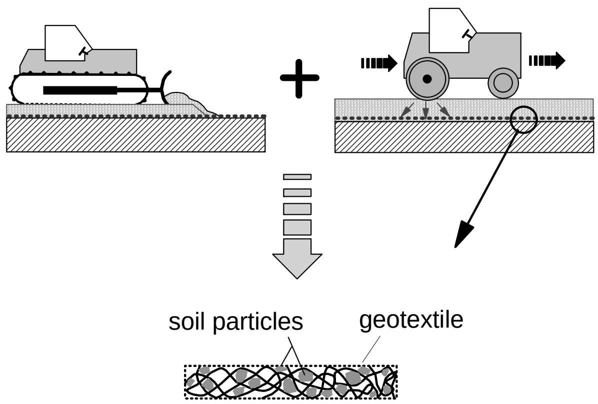
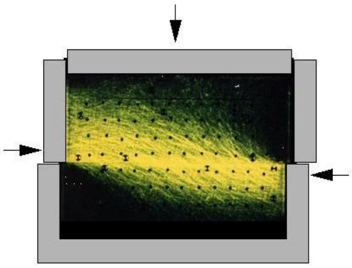
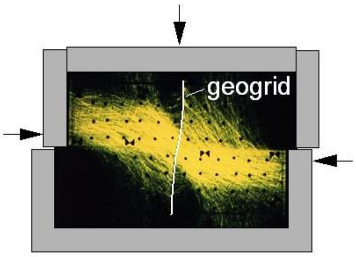


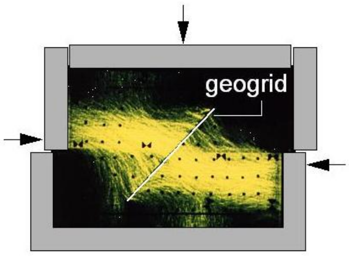
Figure 5. Nonwoven geotextile impregnation by granular material during construction.

The interpretation of the results of direct shear tests on samples with the reinforcement intercepting the shear plane (region C in Fig. 1) is quite complex. This can be verified by test results by Dyer (1985) using photo-elasticity, which showed the influence of the presence of the reinforcement on the state of stress in the soil sample, as presented in Figures 6(a) to (c). In these experiments, crushed glass was used in substitution to soil and photo-elasticity techniques were employed. The normal stress was applied to the sample top by a rigid plate. The photograph in Figure 6(a) was taken during an unreinforced test, whereas those in Figures 6(b) and (c) were taken during tests on samples reinforced by vertical and inclined steel grids, respectively. Bright regions in the photographs are regions of high compressive stresses, whereas dark regions are regions of low stress levels. The pattern of light fringes obtained shows that the presence of the reinforcement changes significantly the state of stress in the sample in a very complex way. In addition, it can also be noticed some interaction with the top and bottom rigid boundaries of the shear box in the reinforced tests, which may influence test results.



(a) Unreinforced.

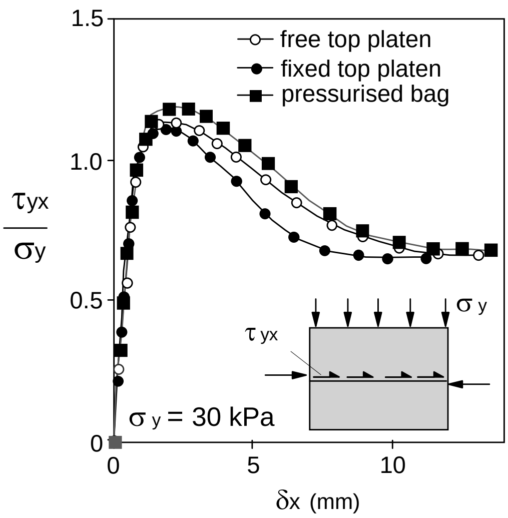


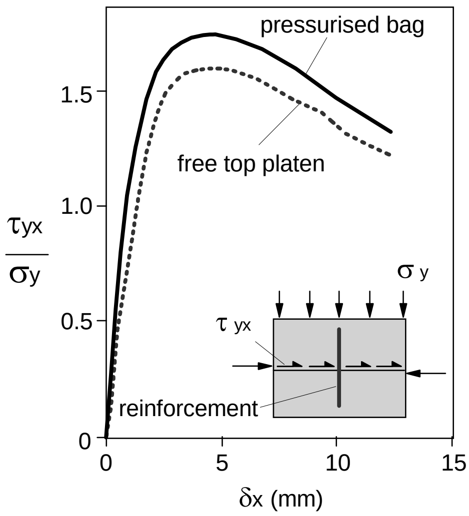


(b) Vertical reinforcement. (c) Inclined reinforcement.

Figure 6. Results of photo-elastic studies in the direct shear test (Dyer 1985).

The influence of the top boundary on direct shear test results can be identified in tests on unreinforced and reinforced sand shown in Figures 7(a) and (b). The shear box used in these tests had dimensions 250mm x 250mm x 150mm. Top boundary conditions tested were a rigid top platen, a top platen fixed to the box top halve and a pressurised rubber bag (Palmeira 1987). In Figure 7(a) the result for the unreinforced sand shows that some differences of peak shear strength can occur and that the tests with a more movable top boundary condition yields to a slightly less fragile behaviour of the sand after peak strength. The influence of the type of the top boundary is more noticeable in the test with a vertical steel grid reinforcement in the sample, as shown in Figure 7(b). This is likely to be in part a result of the interaction between the reinforcement and the rigid boundaries, as indicated in the photo-elastic study (Fig. 6b), and of the different levels of constraint imposed to soil deformation, depending on the top boundary condition considered. Similar differences in peak interface shear strength results were obtained by Hsieh and Hsieh (2003) in direct shear tests on sand-geomembrane interfaces, with the geomembrane at the box middle plane and under different conditions of rigidity of the box top boundary.

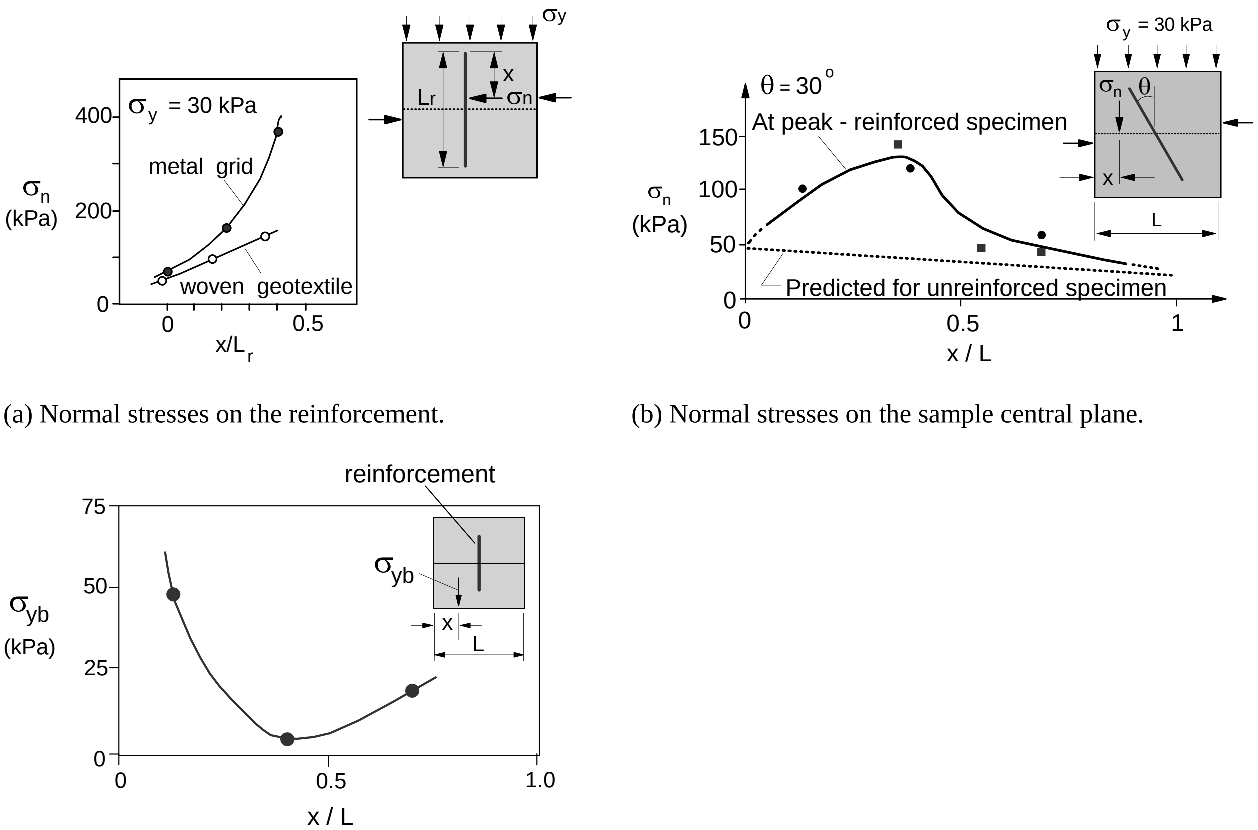




(a) Unreinforced. (b) Reinforced.

Figure 7. Influence of the rigid top boundary on direct shear test results.

Figures 8(a) to (c) show results of large scale direct shear tests carried out on 1 cubic meter reinforced sand samples with and without inclined reinforcements (Palmeira and Milligan 1989b). Total pressure cells were positioned along the reinforcement plane (Fig. 8a), along the failure plane (Fig. 8b) and along the rigid base of the shear box (Fig. 8c). It can be noted the non-uniform stress distributions in these figures. Figure 8(a) shows that the normal stress acting on the reinforcement depends on its type and stiffness, because that stress also depends on the shear load mobilised. Figure 8(b) shows that the presence of the reinforcement causes a significant increase on normal stresses on the shear plane, which will yield to increased strength of the reinforced sample in comparison to the unreinforced one. Figure 8(c) shows the non-linear nature of the normal stress distribution along the box rigid bottom boundary caused by the presence of the reinforcement, which can also be identified in the photo-elastic results presented in Figure 6.



(a) Normal stresses on the reinforcement.

(b) Normal stresses on the sample central plane.

(c) Normal stresses at the box bottom.

Figure 8. Internal total stresses in a large scale direct shear test (Palmeira and Milligan 1989b).

The marked effect of the presence of the reinforcement can also be visualised in the finite element results presented in Figure 9 (Sieira 2003). The soil in this analysis was modelled as an elastic perfectly plastic material. Interface elements between soil and reinforcement were considered as well as the Mohr-Coulomb failure condition for the soil. Figure 9 shows regions of maximum shear stresses in the soil mass at failure for a test under a normal stress of 100 kPa. Both the finite element and the photo-elastic results shown in Figure 6 are consistent and enhance the complex nature of the state of stress in the reinforced soil mass in a test with the reinforcement inclined to the sample central plane.

A summary regarding direct shear tests should point out that:

- It is a good test for determining interface shear strength.
- Boundary conditions may influence test results, particularly for small shear boxes.
- It is difficult to assess soil-geotextile interface shear stiffness, which is relevant for numerical simulations of reinforced soil structures such as walls and steep slopes.
- The interpretation of tests with inclined reinforcements is very difficult.

The complex nature of the results presented above indicates that direct shear tests with inclined reinforcement may provide qualitative data, but the extrapolation of its results to real reinforced structures is still rather limited.

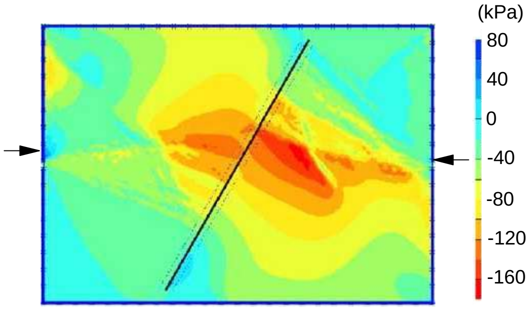


Figure 9. Maximum shear stress distribution in a reinforced sample in a direct shear test–FEM results (Sieira, 2003).

PULL-OUT TESTS

Pull-out tests are relevant for the study of the anchorage strength of reinforcements (Fig. 1), particularly in the case of geogrids. In this type of test one has also to consider the influence of boundary conditions, which is more relevant as the test is not standardised throughout the world. Besides, the interpretation of the results of tests on geogrids is not easy due to the complex geometry of such materials and influence of effects such as soil dilation and reinforcement load-strain-time characteristics, for instance.

Some works in the literature have shown that the conditions of the frontal face of the box can have a marked effect on the test results (Palmeira 1987, Palmeira and Milligan 1989a, Johnston and Romstad, 1989, Farrag et al. 1993, Lopes and Ladeira 1996, Raju 1995, Moraci and Montanelly 2000 and Sugimoto et al. 2001, for instance). Figures 10 (a) to (d) show some typical boundary conditions of the pull-out box found in the literature. In the traditional test arrangement the soil is in contact with the rigid frontal face and it is important that friction along this interface be minimised (Fig. 10a). The reduction of friction along this interface is usually achieved by using layers of plastic films and oil or grease (Palmeira 1987, Abramento 1993, for instance). Alternatives to minimise possible influences of the box frontal face are the use of sleeves, as shown in Figure 10(b) (Farrag et al. 1993, Wilson-Fahmy et al. 1994, Perkins and Cuelho 1999, for instance) or arrangements where the reinforcement length which is effectively tested is distant from the front wall (Fig. 10c, Palmeira 1987). The latter is practical only for tests on geogrids when it is easy to remove some of the frontal transverse members of the grid. Another possibility is to have a flexible front face made of pressurised bags (Fig. 10d) or a movable frontal face (Sugimoto et al. 2001). Whittle et al. (1991) described a pull-out test device where a flexible boundary was used along the rear face of the soil sample.

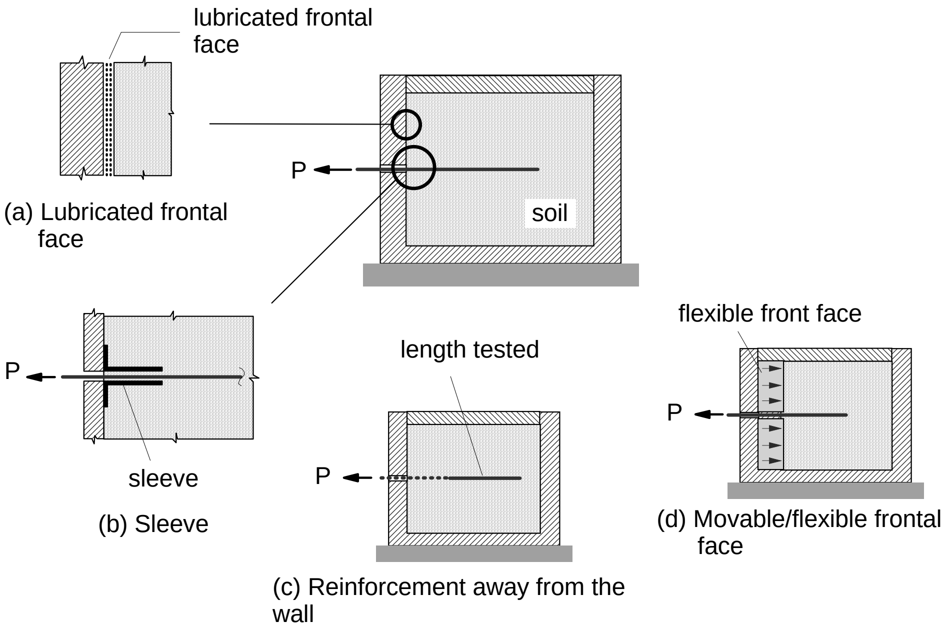


Figure 10. Typical boundary conditions of pull-out tests.

To perform large scale pull-out tests to investigate the influence of boundary conditions is a hard working and time consuming task. Therefore, numerical methods can be used as tools to improve the understanding of some factors that may influence test results. One of the key questions is to what extent the size of the pull-out apparatus may affect the test results. Rather limited experimental data suggests that the size of the apparatus may have a significant effect on the results obtained (Dyer 1985, Palmeira and Milligan 1989a). This influence is also suggested by results from numerical analyses (Dias 2003). Figure 11 shows results of pull-out tests on a reinforcement buried in sand obtained using the finite element method. In the simulations the box height was varied. Soil was model as an elastic-plastic material and the reinforcement as a linear elastic material. The lengths of the box (2m) and of the reinforcement (0.5m) were kept constant. A rigid and lubricated box frontal face with 6^{o} interface friction angle was assumed in the simulations as well as along the other interfaces of the apparatus. The box height was varied between 0.3m and 1 m. It can be noted that the lowest box height yielded a stiffer pull-out response and a higher maximum pull-out load. Little influence of the box height was observed for heights greater than the reinforcement length. Similar conclusions were obtained by Farrag et al. (1993) and by Lopes and Ladeira (1996) in pull-out test experiments where the thickness of the soil was varied. As the reinforcement length tested in pull-out tests is typically smaller than 1m, the numerical study described above and experimental results suggest that under these conditions the soil sample height should be greater than 0.6m.

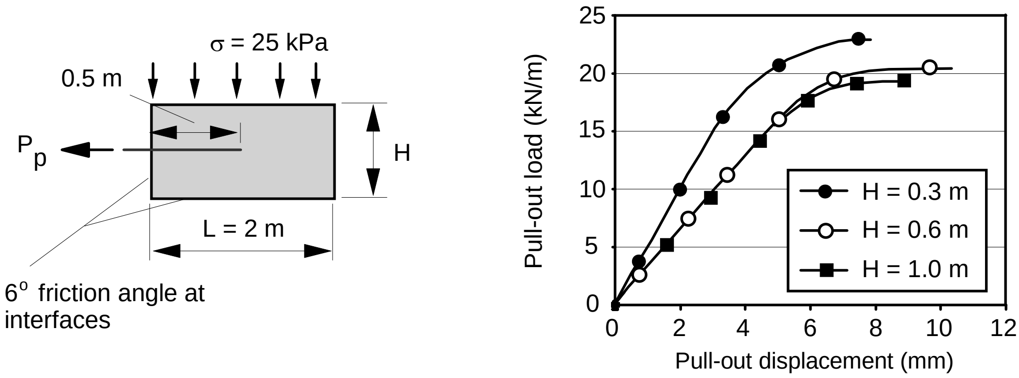


Figure 11. Influence of the pull-out box dimensions – numerical analyses (Dias 2003).

The finite element method can also be used to assess the influence of the presence of a sleeve at the box front wall in an example of a test with a reinforcement with a tensile stiffness (J) of 200 kN/m in a hypothetical box with the dimensions shown in Figure 12 (Dias 2003). In this case, the soil was also modelled as an elastic-plastic material and the reinforcement as a linear elastic material. Figure 12 shows the comparison between predictions of test results with no sleeve and a lubricated frontal wall and with sleeves 15 cm and 30 cm long. The results in this figure show that for the conditions analysed the presence of the sleeve yielded maximum pull-out loads greater than that observed in the case of a lubricated frontal face. The sizes of the sleeves used did not influence significantly the maximum pull-out load obtained. In contrast, Farrag et al. (1993) found a maximum pull-out load in a pull-out test with a 20 cm long sleeve approximately 20% greater than that obtained in a test with a sleeve 30 cm long. Lopes and Ladeira (1996) found a maximum pull-out load in a test without sleeve (and a smooth, but apparently not lubricated frontal face) 10% greater than that obtained in a test with a 20cm long sleeve. Thus, these contrasting results suggest that a more comprehensive study on how to minimise the influence of the front box wall on pull-out test results is required.

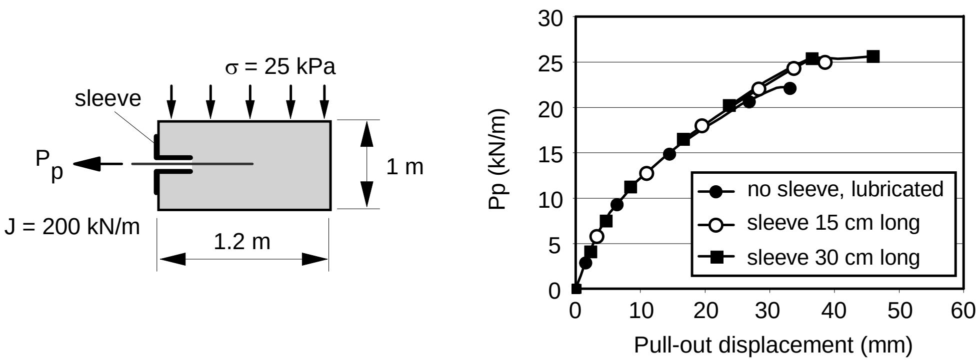


Figure 12. Influence of the presence of sleeves at the box frontal face – numerical analyses.

Another possible arrangement without the use of sleeve consists of placing the reinforcement away from the wall, as show in Figure 13 (Palmeira 1987). Geogrids (particularly those with well defined square or rectangular apertures) can be tested cutting unnecessary transverse members and isolating the longitudinal members from the surrounding soil by pipes, for instance. In Figure 13 the result of the test on 1 cubic meter soil (dense Leighton Buzzard sand) sample with a steel grid away from the front wall (first transverse member 300mm away from the box face) is compared to the traditional test arrangement with a lubricated frontal face. It can be observed that the difference between maximum pull-out loads is small, but the arrangement with the grid away from the wall presents greater and increasing pull-out loads for pull-out displacements above 7mm. This is in part due to the fact that the first bearing member in this type of arrangement keeps on finding undisturbed soil ahead and the testing length of the grid remains the same, whereas in the traditional arrangement transverse members leave the soil sample and the length of the grid effectively tested decreases during the test.

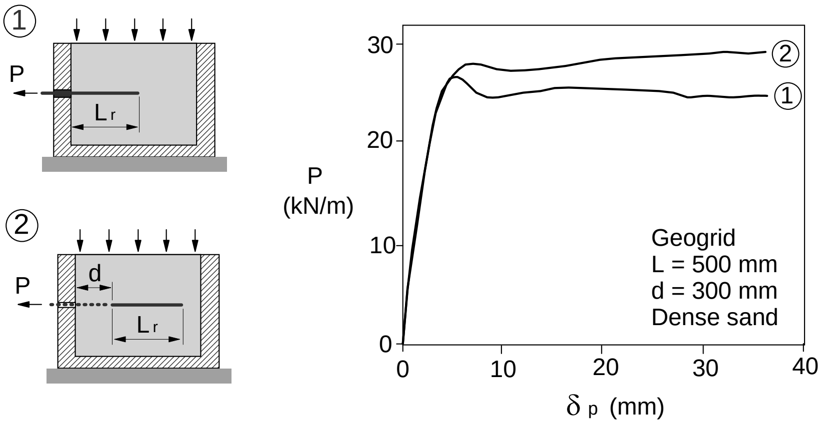


Figure 13. Results of tests with the reinforcement away from the box frontal face (Palmeira 1987).

Sugimoto et al. (2001) performed tests on polymeric grids buried in sand using a large pull-out test apparatus with a movable frontal face, as shown in Figure 14. This figure also presents comparisons between results of tensile strain distributions along the geogrid length for tests with movable and fixed frontal faces under 49kPa normal stress and for a pull-out load of 15.4kN/m. Different distributions of strains along the geogrid length can be seen, depending on the mobility of the frontal face considered.

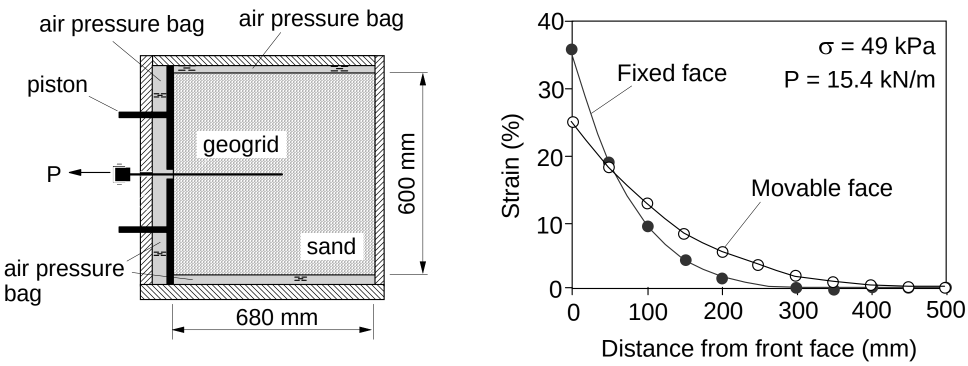
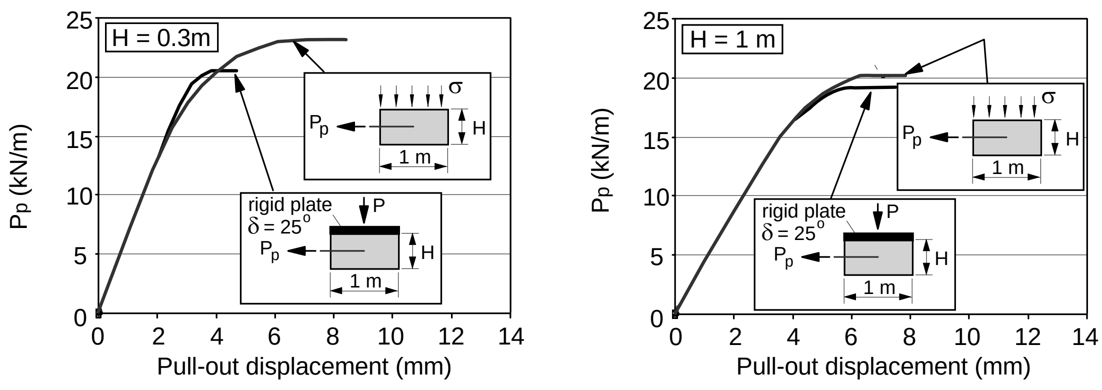


Figure 14. Results of tests obtained in a pull-out box with a movable frontal face (Sugimoto et al. 2001).

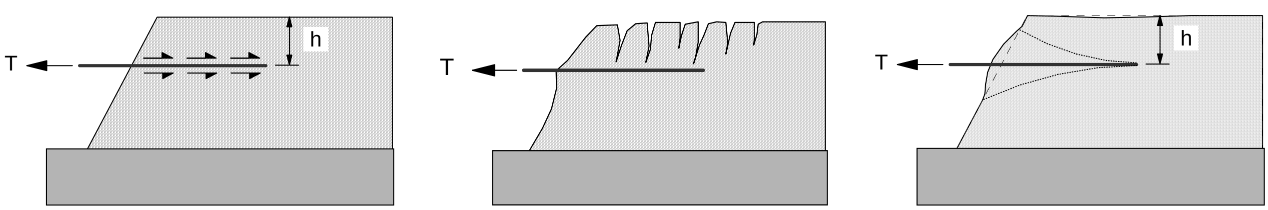
Numerical simulations also indicate that the rigidity of the top boundary influences pull-out test results, particularly for low height samples. Figure 15 shows numerical predictions of pull-out forces versus pull-out displacements in a conventional box arrangement and two box heights (0.3m and 1m) obtained by Dias (2003). The soil was modelled as an elastic-plastic material and the reinforcement as a linear elastic material. Interface elements were used at the soil-reinforcement interface. Rigid and flexible top boundaries were simulated. The results show that the higher the sample the less the influence of the top boundary rigidity on the test results. These numerical results and experimental ones (Palmeira and Milligan 1989a, Farrag et al. 1993, Lopes and Ladeira 1996) clearly suggest that large pull-out boxes should be preferred.

Field pull-out tests are alternatives for those in the laboratory (Bergado et al. 1992, Bakeer et al. 1998, Becker 2006, for instance). This type of test is usually difficult to perform, because of less controlled operational conditions in the field in comparison to those in the laboratory. Clamps for the reinforcement and reaction for the applied pull-out load have to be carefully designed. In tests in real reinforced structures one has also to consider the possible influence of the conditions of the wall frontal face (if any) on the test results. Tests with the reinforcement buried in embankments can be performed, as schematically shown in Figure 16(a). However, due care should be taken in the interpretation of the results from these types of tests because of the usually low stress levels on the geosynthetic required for the test to be practical. Besides, different and some times unrealistic failure mechanisms may occur, depending on the thickness of soil above the geosynthetic layer, uniformity of fill material properties and boundary conditions, as shown in Figures 16(b) and (c).



(a) H = 0.3m (b) H = 1.0m

Figure 15. Influence of the box top boundary rigidity on the test results – numerical analyses (Dias 2003).

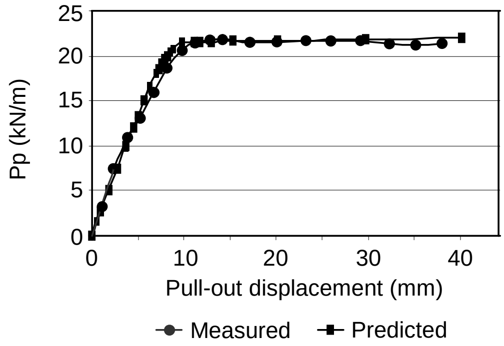


(a) (b) (c)

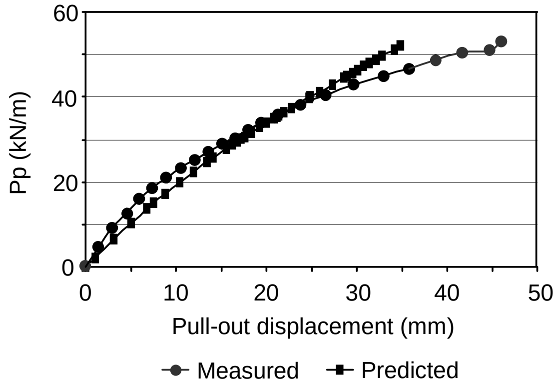
Figure 16. Pull-out tests in embankments.

Analytical and numerical methods can be employed for the study of soil-reinforcement interaction (Palmeira 1984, Abramento 1993, Bergado and Chai 1994, Sobhi and Wu 1996, Madhav et al. 1998, Gurung and Iwao 1999, Gurung 2001, Perkins 2001, Palmeira 2004, for instance). Abramento (1993) discussed the use of the shear-lag model for the evaluation of interaction between soil and reinforcement and his study approached the use of planar and continuous reinforcing materials. The Finite Element Method can also be used as a tool for the back-analysis of the results of pull-out tests (Wilson-Fahmy et al. 1994, Yogarajah and Yeo 1994, Perkins 2001, Dias 2003). Figures 17(a) to (c) show some comparisons between measurements and predictions by the finite elements method of pull-out tests on a geotextile and on a geogrid (Dias 2003). The tests modelled were carried out on 1 m^3 dense sand samples. The woven geotextile was made of polyester with a tensile stiffness of 4000 kN/m. The geogrid was made of high density polyethylene, with average tensile stiffness of 550kN/m. The soil was modelled as an elastic-plastic material with Mohr-Coulomb failure criterion. The reinforcements were assumed as linear elastic materials. The geometry of the geogrid transverse members was considered in the grid modelling and its longitudinal members were modelled as equivalent planar elements (plane strain conditions). Figures 17(a) and (b) show the results in terms of pull-out loads versus pull-out displacements and Figure 17(c) shows the predicted and measured tensile strains distributions along the grid length. These results show that, in spite of some simplifying hypotheses usually assumed in this type of analysis (value of interface shear stiffness, for instance), it is possible to make predictions fit reasonably well the measurements, particularly for geotextiles. However, as is common in this type of analysis, this may not necessarily be achieved with the actual or expected values of some of the input parameters. In the case of geogrids, if the grid is assumed as an equivalent rough planar reinforcement, predictions may deviate from measurements, depending on the grid geometrical characteristics and soil type. Besides, when relevant, the load-time-strain relationship for the inclusion may also affect the results, if this was only approximately accounted for, which may in part explain some deviations between results in Figure 17(b).

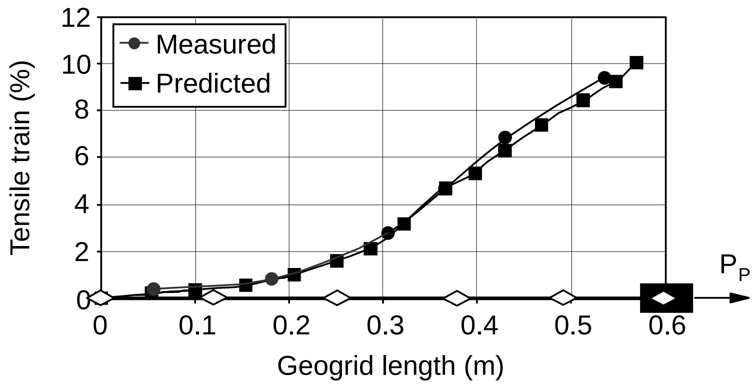
Because the large pull-out box used in the tests described above had total pressure cells along its frontal face, it was also possible to compare predicted and observed normal stresses along that boundary. The rigid pull-out box internal frontal face was lubricated with double layers of plastic films and grease, yielding to an interface friction angle of 6°. Figure 18 shows results of predictions and measurements for a test on a HDPE geogrid in terms of the increments of normal stress ($\Delta\sigma_x$) along the box internal frontal face (y is the elevation along the box face and h is half of the box height) normalised by the average shear stress (τ_b) along the reinforcement length (Palmeira and Dias 2007). Again, a reasonable comparison between predicted and observed normalised normal stresses can be observed.



(a) Load-displacement curves for a woven geotextile.



(b) Load-displacement curves for a HDPE geogrid.



(c) Tensile strain distribution along the geogrid length.

Figure 17. Predicted and observed pull-out test results.

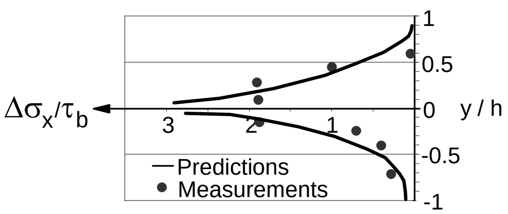
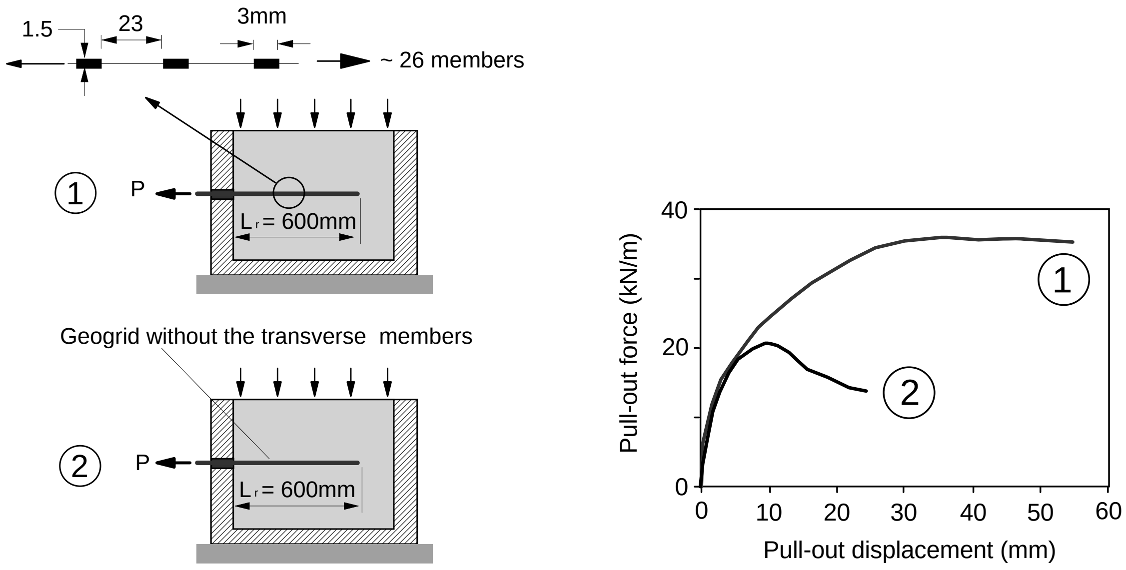


Figure 18. Predicted and observed normal stresses along the box frontal wall (Palmeira and Dias 2007).

Finite Element Method analyses usually assume a geogrid as a continuous equivalent rough planar reinforcement. However, the pull-out response of this type of reinforcement is fundamentally dependent on shape and geometrical characteristics of the grid, with particular reference to the transverse bearing members. Figures 19(a) and (b) show results of pull-out tests performed by Teixeira (2003) on a commercially available geogrid, made of polyester, with apertures 23 mm x 23 mm, buried in dense granular material. Firstly, tests were performed on the geogrid with its original geometrical characteristics. Then, the transverse members were removed and only the longitudinal members were tested (Fig. 19a). It can be observed in Figure 19(b) that the transverse members are responsible for a significant fraction of the maximum pull-out load of the geogrid, but friction along longitudinal members is equally important for that matter. Longitudinal members alone show a strain softening behaviour after peak, while the entire grid shows a rather constant maximum pull-out load with increasing pull-out displacements.



(a) Types of tests carried out.

(b) Test results.

Figure 19. Influence of the grid bearing members on the pull-out response of a geogrid (Teixeira, 2003).

Other aspects that influence the grid response under pull-out are the shape and bending stiffness of the transverse members. Figure 20 shows a photograph of the bended shape of a grid transverse member at the end of a large scale pull-out test (Santos 2007). For transverse members with low bending stiffness a progressive mobilisation of bearing resistance will take place as the member bends under increasing pull-out loads. Brown et al. (2007) also discuss the importance of transverse members bending stiffness on the performance of geogrid reinforced ballast. This influence has still to be properly studied and quantified.

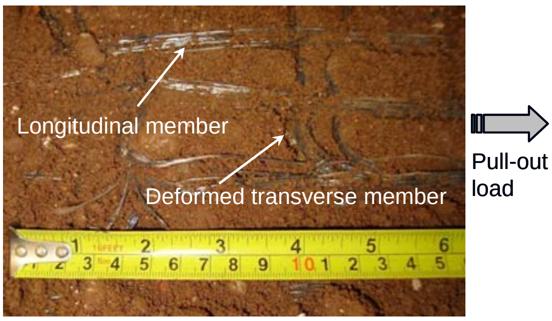


Figure 20. Bended grid transverse member at the end of a pull-out test (Santos 2007).

As shown above, interaction between soil and geogrid is quite complex. If the soil is assumed as a continuum, geogrid pull-out could be represented by the simultaneous pull-out of a series of plate like elements buried in the soil. The assumption of the soil particles fixed in between transverse members while the grid is being pulled is certainly unrealistic. Jewell at al. (1984) proposed upper and lower bound theoretical solutions for the bearing stress at the transverse member normalised by the vertical stress as a function of the soil friction angle. These upper and lower bound solutions are shown in Figure 21 and were derived from analyses of plates buried in a homogeneous continuum. Moraci and Gioffrè (2006) found good agreement between predictions by theoretical solutions such as those proposed by Jewell at al. and results from large scale pull-out tests on grids with large spacing between transverse members and little scale effects.

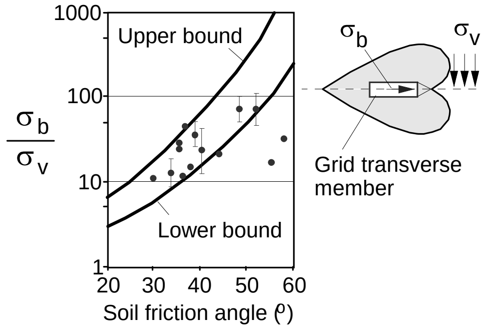


Figure 21. Bearing stresses on isolated grid transverse members (Palmeira and Milligan 1989a).

However, the relative size of the soil particle with respect to the grid transverse member thickness will influence the bearing stress ratio. The data points in Figure 21 are experimental results from pull-out tests on isolated transverse members and the scatter observed is mainly due to effects of the relative sizes between transverse member and soil particles (Palmeira and Milligan 1989a). Thus, solutions based on continuum may not be directly extrapolated to a particulate medium where the grid transverse member and the soil particles have similar dimensions (note the logarithmic scale of the graph in Fig. 21).

The influence of the relative sizes of soil particle and grid transverse member can be clearly visualised in Figure 22, where the normalised bearing stress is plotted against the transverse member thickness normalised by the average soil particle diameter. The results presented in this figure are those from tests on isolated transverse members with circular, square, hexagonal and rectangular cross-sections buried in dense sands or crushed glass. The result of a test on a member of a polymeric grid (hexagon like cross-section) is also shown in this figure. Figure 22 shows that bearing stress depends on the transverse member shape with square (or rectangular) members providing slightly greater bearing strengths. The results also show that the normalised bearing strength starts to be independent on the soil particle size only for ratios between member thickness and average particle diameter above 12.

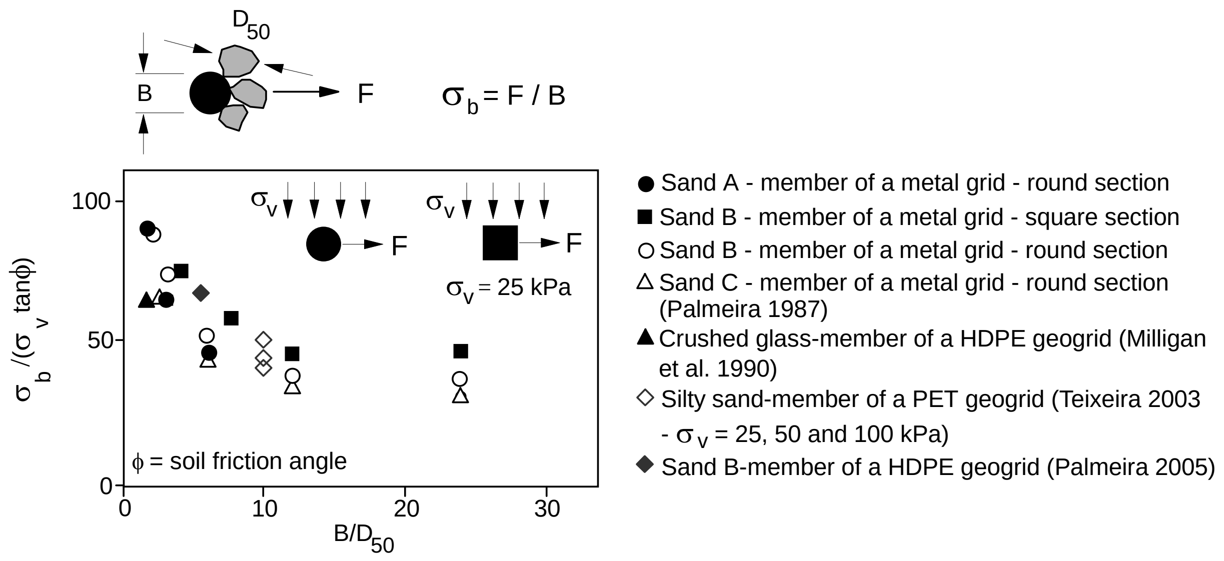
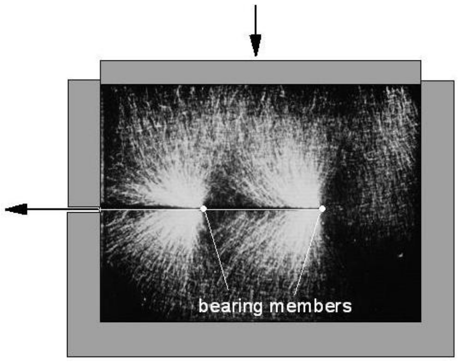


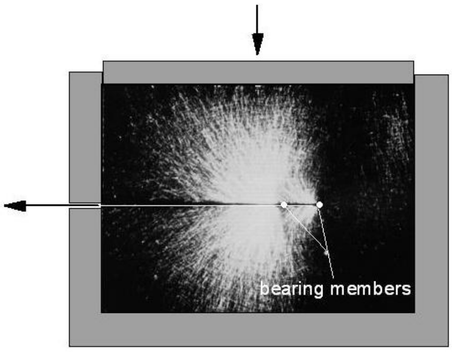
Figure 22. Results of pull-out tests on isolated transverse members with different cross-sections (modified from Palmeira and Milligan 1989a).

Another particular aspect of the interaction between soil and geogrid is the possibility of interference between grid transverse members. The discrete interaction between grid transverse members and the surrounding soil is clearly visible in the results of photo elastic studies conducted by Dyer (1985), some of them reproduced in Figures 23(a) and (b). These results show that the load sharing between transverse members of a steel grid is uniform only if the members are sufficiently apart from one another (Fig. 23a). As the distance between them is reduced (Fig. 23b) significant non uniformities in the distribution of bearing loads among transverse members can occur. This non-uniform load sharing is a result of interference between grid transverse members.

Figures 23(a) and (b) showed grids with only two transverse members. For a longer grid the interaction with soil is more complex but the photo-elastic results still show that the load sharing between transverse members is not uniform, as depicted in Figure 24 (Dyer 1985). In this figure the contour of the dark region behind each transverse member has been highlighted by a continuous line. These dark regions are regions of low stress levels caused by the movement of the transverse members and it can be seen that they vary in size along the grid length. The movement of a rigid grid in the soil leaves loose soil regions at the rear of each transverse member. Ahead of the member the soil is subjected to a passive state of stress whereas behind it an active state of stress is reached, creating the loose soil regions. This state of stress is represented by the dark regions in the photo-elastic results shown in Figure 24. The size of this region and its proximity to the next member will influence the pull-out load sharing among transverse members. Besides, pull-out tests on isolated bearing members in dense sands with large ratios between member thickness and sand particle diameter have shown that the extension of the soil mass ahead of the member affected by the failure mechanism can be as much as 6 times the transverse member thickness (Palmeira and Milligan 1989a). In situations where the soil particles and the transverse members have similar dimensions, contact force chains can cause loading spreading over a large region and be quite long (Fig. 23a and McDowell et al. 2006), yielding to much more complex interference conditions.



(a) Large spacing between transverse members.



(b) Short spacing between bearing members.

Figure 23. Interference between grid transverse members (Dyer 1985).

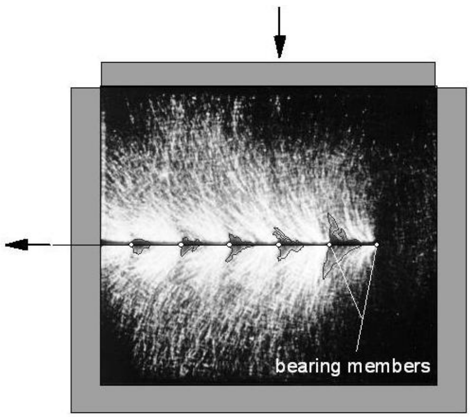


Figure 24. Photo-elastic results of pull-out test on a steel grid (Dyer 1985).

For a grid with large apertures (small solid fraction per unit area), the larger the spacing between transverse members the less the influence of interference between them. Under these conditions, interference can be quantified by the Degree of Interference (Palmeira 1987, Palmeira and Milligan 1989a), which compares the actual pull-out strength of a geogrid with the maximum pull-out strength if there was no interference between transverse members, or as if the spacing between these members was very large (negligible skin friction between soil and grid). Figure 25 shows the definition of the Degree of Interference (DI) and results of degree of interference against the ratio between transverse member spacing (S) and member diameter (B) obtained from tests on steel grids with round transverse members buried in dense sand. The results suggest that negligible or no interference between transverse members will occur only for S/B ratios above 40. In this case, each member of the grid would behave as it was isolated. The figure also shows predictions of DI values for round and square transverse members by the Finite Element Method (Dias 2003). It is interesting to note that the same trend of reduction of DI with the increase of S/B is predicted by the Finite Element Method. It should be pointed out that the results presented in Figure 25 were obtained for stiff grids.

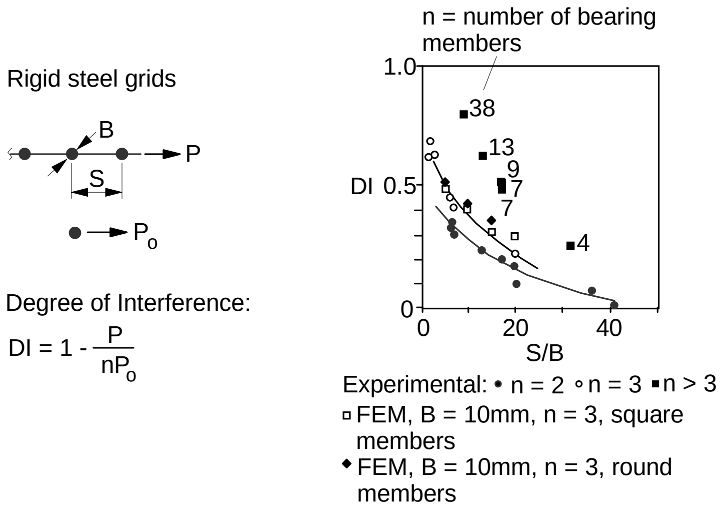


Figure 25. Degree of interference versus the ratio between transverse member spacing and member thickness.

For extensible grids the evaluation of the degree of interference is even more complex due to the non uniform distribution of tensile strains along the grid length. Milligan et al. (1990) found more noticeable non uniform distributions of bearing loads in a polymer grid in a photo-elastic study similar to that carried out by Dyer (1985). This non-uniformity was certainly enhanced by the lower tensile stiffness of the polymer grid and the higher stress level used in the tests in comparison with the high tensile stiffness of metal grids. Nevertheless, nowadays increasingly stiffer polymeric grids are being produced and the results of Figure 25 may be extrapolated to some extent to these materials.

A consequence of the influence of grid geometrical characteristics, interference between transverse members and the assumption of the grid as a planar continuous element is that the apparent soil-geogrid friction coefficient becomes dependent on the length of the geogrid, which complicates the estimate of soil-geogrid bond for design purposes. The results by Moraci and Gioffrè (2006) presented in Figure 26 show the variation of the apparent friction coefficient of a geogrid as a function of the grid length and of the normal stress. It can be seen that the shorter reinforcement was the one presenting the largest apparent friction coefficient. It is also shown that the lower the stress level the higher the friction coefficient, in part due to a greater dilative response of the dense sand under low stresses. These results suggest that in a real reinforced soil wall, for instance, different apparent friction coefficient values should be considered in shallow and deep anchorage lengths.

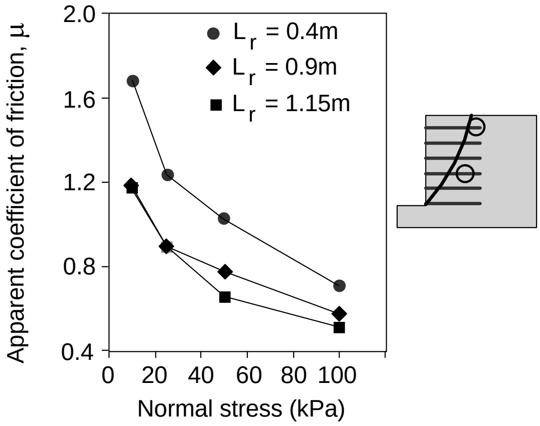


Figure 26. Grid apparent friction coefficient versus normal stress (modified from Moraci and Gioffrè 2006).

It would be interesting to evaluate the non uniform distribution of bearing loads among grid transverse members. However, the results presented so far have shown that the particulate nature of the interaction between granular soils and typical grids makes modelling of soil-grid interaction very complex. An additional difficulty is that skin friction between soil and grid may be a significant part of the bond strength between these materials and the variety of shapes and geometrical characteristics of available grids adds to the complexity of the problem. The use of finite element analysis may provide a good overall understanding of the response of a geogrid if the grid is substituted by an equivalent rough sheet, but limitations in the simulations do exist, like the value of interface shear strength and stiffness to be used in the analyses, for instance. Finite element analyses have shown that soil-inclusion shear stiffness varies with the stress level (Perkins 2001, Dias 2003). The use of the discrete element method (Cundall and Strack, 1979, McDowell et al. 2006) is certainly very promising, but simulations are very time-consuming and modelling of realistic problems requires more powerful computers than those currently available. To assess the distribution of loads among grid transverse members a simple alternative approach would be to model the geogrid as a discrete material consisting of a succession of bearing elements (Palmeira 1984).

The model shown in Figure 27 can be used for the investigation of load distribution along grid transverse members and, in particular, for the back-analyses of results of pull-out tests on geogrids (Palmeira 2004). The model takes into account a friction law for the interaction between soil and longitudinal grid members. It also considers the bearing force-member displacement relationship and, when relevant, the load-strain-time relationship for the geogrid can be accounted for. Based on these assumptions, equilibrium equations can be derived and an iterative procedure used to obtain the pull-out response of the grid and the load distribution among grid transverse members (Palmeira 2004). By trial and error different bearing load-displacement relations can be used to assess non uniform load distribution among transverse members of geogrids. In such trials the result obtained from a pull-out test on an isolated member (Fig. 27) can be used as reference. In the actual grid the load-displacement curve obtained for the isolated member would be valid only for the first transverse member, even so if it was distant enough from the pull-out box front wall or sleeve. The load-displacement curves for the other grid members can be established based on arbitrary reduction functions applied to the curve obtained for the isolated member (Palmeira 2004), depending on the transverse member considered. The choice of the function is arbitrary and the best choice will be the one for which the predictions by the model fit the best the load-displacement curve and the tensile strain distribution along the grid length observed in the pull-out test on the actual grid.

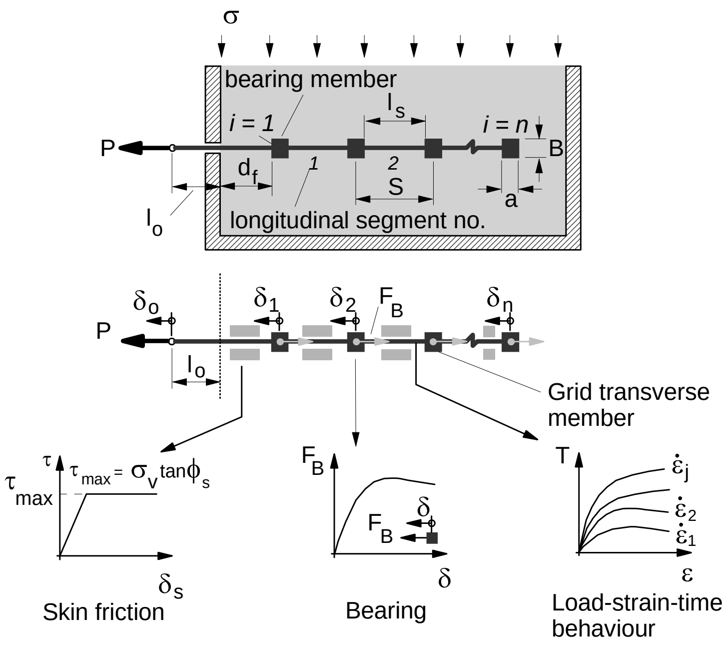


Figure 27. Discrete approach for geogrid modelling.

The model described above is useful for the back-analysis of the results of pull-out tests on geogrids, particularly those with simpler geometries. However, it is certainly not practical for the prediction of geogrid pull-out responses in general, because the evaluation of the load-displacement relationship for isolated transverse members is not straightforward. Besides, some mechanism of interference between transverse members has to be assumed, unless the effects of interference can be neglected.

The model in Figure 27 and the results of some large scale pull-out tests carried out in the apparatus shown in Figure 28 (Palmeira 1987) can be used in the back-analysis of the non uniform distribution of loads in grid transverse members. The testing equipment consisted of a large pull-out box capable of accommodating 1 cubic meter soil samples. A pressurized bag provided the normal stress at the top of the sample. The internal frontal face of the box was lubricated with double layers of plastic films and oil. The frontal face had total pressure cells to measure normal stresses transmitted to this boundary during the tests and dense and uniform Leighton Buzzard sand was used in the experiments. Woven geotextiles, metal and polymeric grids were tested. During the tests, tensile strains were measured along the length of polymeric grids.

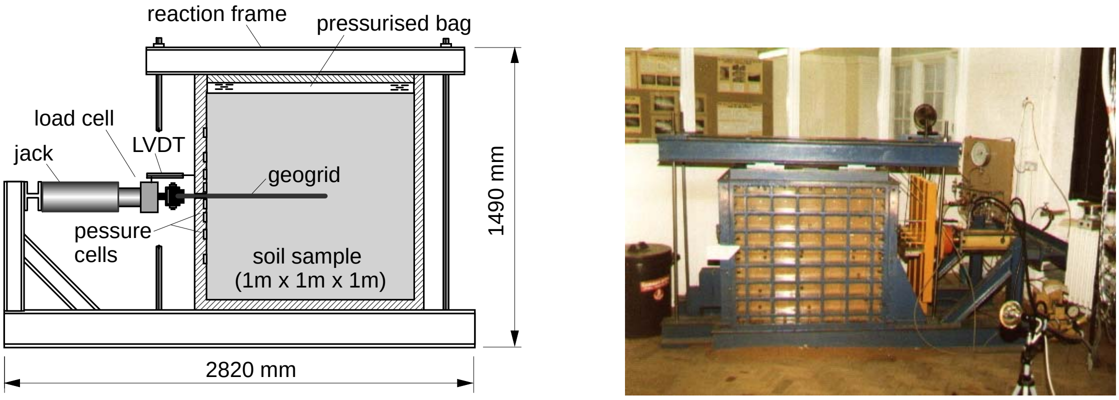
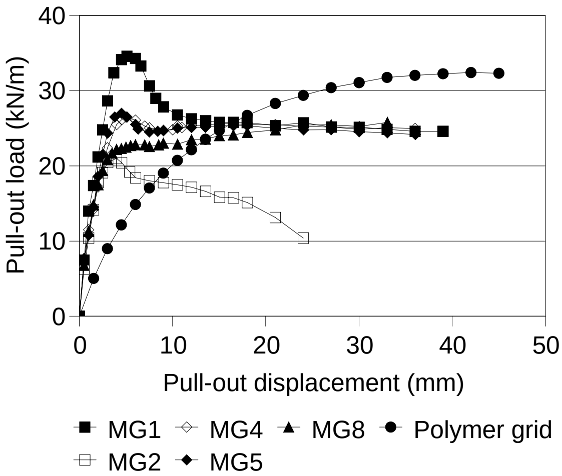
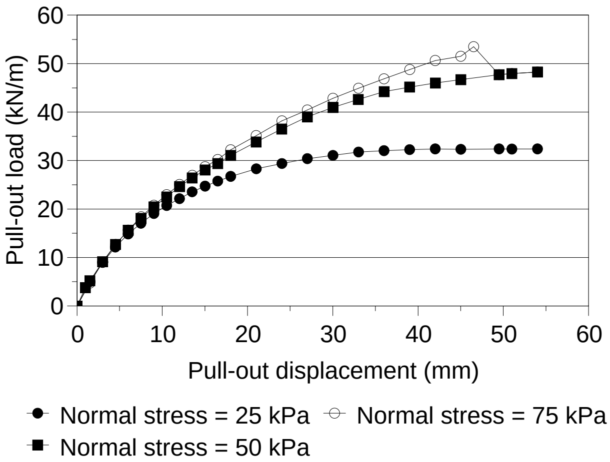


Figure 28. Large scale pull-out test apparatus.

Figures 29(a) and (b) summarise some of the results obtained in terms of pull-out loads versus pull-out displacements. Figure 29(a) shows results of tests on rigid metal grids (code MG) with round transverse members in a square or rectangular pattern and on a polymeric grid. The response of the metal grids was much stiffer than that of the polymeric grid, but significant strain softening behaviour was observed for the metal grid which performed the best in comparison to the response of the polymeric grid, although both presented similar values of maximum pull-out loads. Figure 29(b) shows results obtained in tests with the polymeric grid under different normal stresses, where it can be seen that the higher the normal stress the more extensible the response of the geogrid.



(a) Pull-out force versus pull-out displacements.

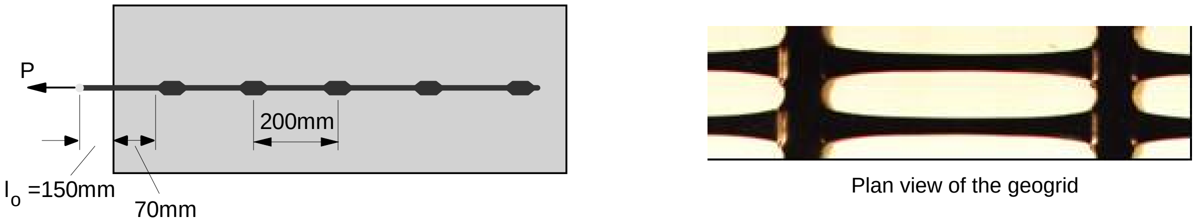


(b) Results for polymeric grids under different normal stresses.

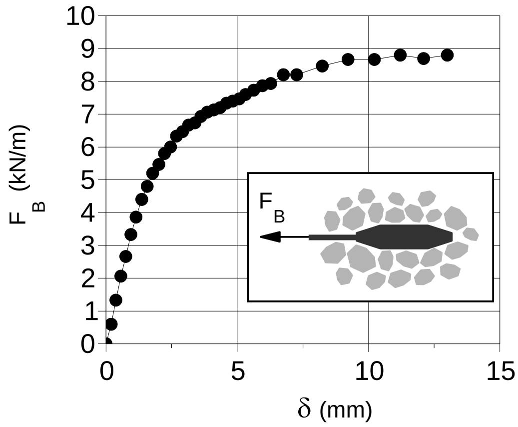
Figure 29. Some results of large scale pull-out tests on metallic and polymeric grids (Palmeira 2004).

The input data for the theoretical model described before is shown Figures 30(a) to (c). The dimensions and geometrical characteristics of the polymeric grid tested, made of high density polyethylene, are shown in Figure 30(a). Figure 30(b) presents the result of a pull-out test on an individual isolated transverse member of the polymeric geogrid. The member has an approximately trapezoidal cross-section shape. The load-strain-time relationship for the geogrid is presented in Figure 30(c), where the results from tensile tests under different strain rates are shown (McGown 1982).

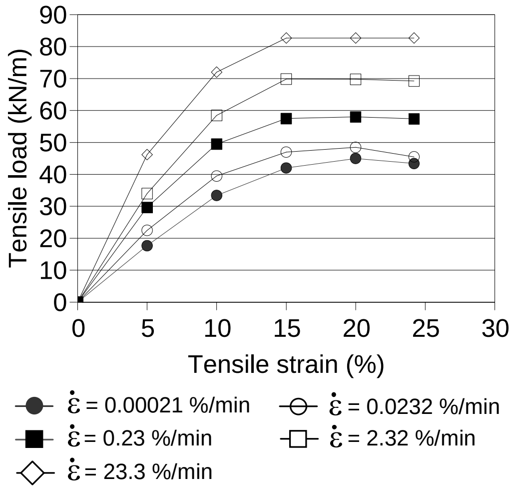
Based on those input parameters, Figures 31(a) to (c) present comparisons between the results of a large pull-out test on a polymer grid under 25 kPa normal stress and a test speed of 0.5 mm/min and the best fit prediction based on an arbitrarily distribution of forces on each transverse member (Palmeira 2004). Figure 31(a) shows a reasonable agreement between predicted and observed geogrid pull-out force versus pull-out displacement curves. A good agreement is also observed for the tensile strain distribution along the grid length at maximum pull-out force, as shown in Figure 31(b). Figure 31(c) presents the distribution of loads among the transverse members for the best fit hypothesis and a non uniform load distribution can be noticed. This non uniform load distribution is caused by interference between transverse members and by different mobilisations of tensile strains along the grid length.



(a) Geometrical characteristics of the geogrid.



(b) Force-displacement relationship for an isolated transverse member.



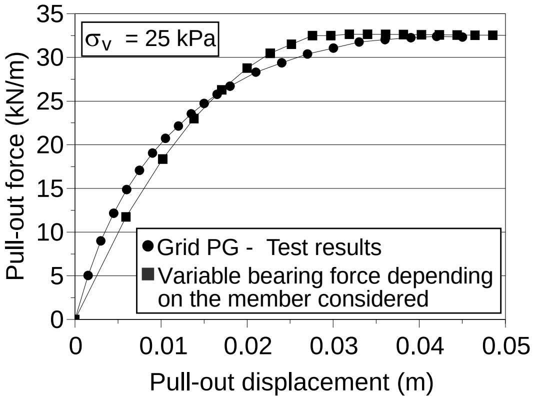
(c) Load-strain-time relationship for the geogrid (McGown 1982).

Figure 30. Input data for the simulation using the discrete approach.

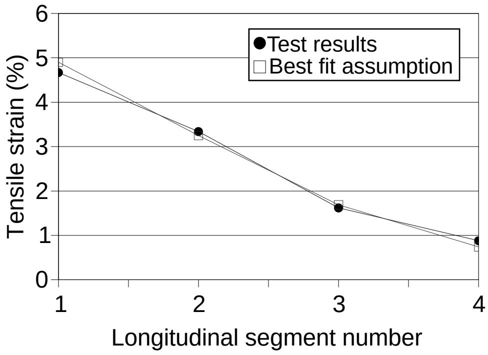
If the same exercise is repeated for tests under higher stress levels, results such as the ones presented in Figures 32(a) and (b) are obtained at maximum pull-out force. The non uniformity of the bearing force distribution increases and the maximum bearing forces may not necessarily act on the first transverse member but on subsequent members. These results show the level of complexity of the mechanism of interaction between soil and geogrid in a pull-out test box and are consistent with the results from photo-elastic studies.

At this stage, a summary of the conclusions on pull-out tests should point out that:

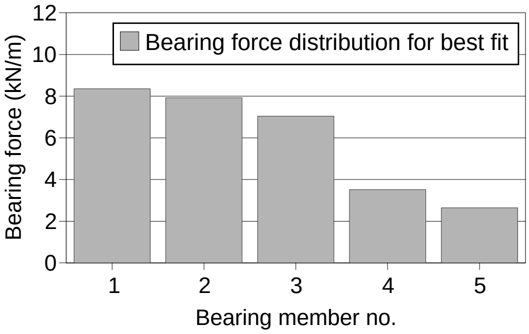
- The pull-out test is useful for determining anchorage strength of geogrids.
- Boundary conditions may influence test results.
- Regarding minimising the influence of the wall frontal face, a sleeve or a lubricated internal face can be used, but numerical and experimental results suggest some differences on pull-out responses and maximum pull-out loads depending on the solution adopted. Some authors have reported close values of maximum pull-out loads in tests with well lubricated box frontal face, sleeve or with the reinforcement away from the frontal face. However, the use of a sleeve seems to provide greater confidence on the less influence of the box frontal wall on the test results. The appropriate length of the sleeve is likely to be dependent on the roughness (which should be reduced anyway) of the box wall, height of the sample and length and type of reinforcement. The results available suggest that for the typical dimensions of large scale pull-out devices the sleeve length should not be less than 30cm.
- Large scale tests should be preferred, particularly because increasing the scale of the test reduces the detrimental effects of the boundaries. Experimental and numerical results suggest that for typical reinforcement lengths (< 1m) in pull-out tests the height of the soil sample should not be smaller than 0.6m.
- The lack of practical and accurate general prediction methods and the variety of types and geometrical characteristics of geogrids available enhance the importance of conducting high quality pull-out tests to evaluate bond between soils and geogrids.
- Numerical models can be useful tools for the understanding of the pull-out behaviour of geogrids.



(a) Pull-out force versus pull-out displacement.

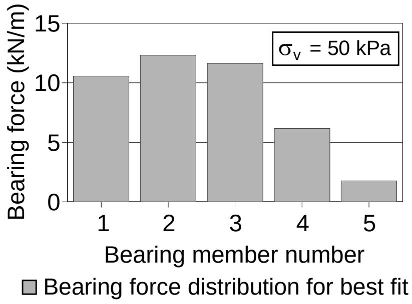


(b) Comparison between strain distributions along the grid length.

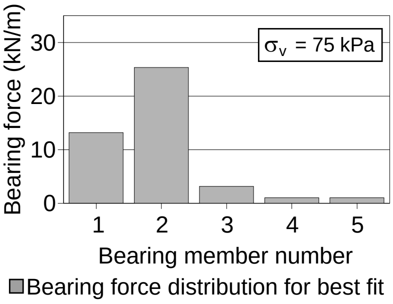


(c) Distribution of forces among grid transverse members.

Figure 31. Predicted and observed pull-out test results using the discrete approach (Palmeira 2004).



(a) Normal stress of 50 kPa.



(b) Normal stress of 75 kPa.

Figure 32. Non uniform distributions of bearing loads among geogrid transverse members (Palmeira 2004).

IN-SOIL TENSILE TEST

The in-soil tensile test is relevant for geotextiles, particularly nonwovens. The influence of confinement on the mechanical behaviour of mechanically bonded nonwoven geotextiles has long been acknowledged thanks to the pioneer work by McGown et al. (1982). Figure 33 shows schematically the test device and some results obtained by McGown et al. in tests on nonwoven geotextiles. Since then, several researchers have also investigated the load-strain behaviour of geotextiles under confinement (Leshchinsky and Field 1987, Siel et al. 1987, Wu and Arabian 1988, Kokkalis and Papacharisis 1989, Ling et al. 1992, Gomes 1993, Azambuja 1994, Tupa 1994, Boyle et al. 1996, Palmeira 1996, Palmeira et al. 1996, Helwany and Shih 1998, Mendes 2006 and Mendes et al. 2007). Confinement increases interlocking and friction among geotextile fibres, yielding to a stiffer response of the geotextile. If in significant amounts, the soil particles that intrude the fibre matrix restrain fibre stretching, also contributing to the increase of geotextile tensile stiffness. In some of the testing apparatus reported in the literature, the geotextile ends are stiffened by impregnation with epoxy resin. So, only the central part of the specimen suffers significant deformation. It is important to point out that in this type of test arrangement friction between geotextile and confining soil also takes place and this influences the magnitude of tensile load and tensile stiffness measured.

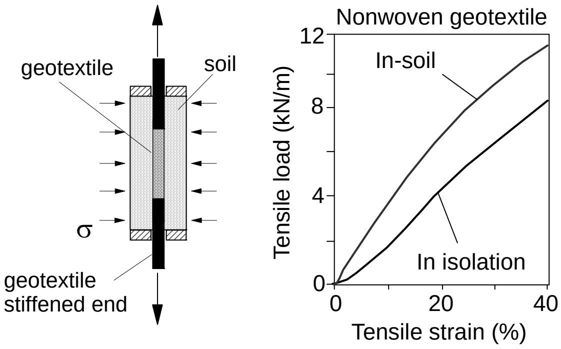


Figure 33. Effects of confinement on geotextile stiffness (McGown et al. 1982).

Figure 34 shows results of in-soil tensile tests in terms of secant tensile stiffness versus tensile strain for tests with different soil types and a nonwoven needle punched geotextile using a test arrangement similar to that shown in Figure 33 (Palmeira et al. 1996). Soils varied from silts to coarse sand. The result for a test with a lubricated rubber membrane confining the geotextile is also presented and this test provided a lower bound for the test results. In the test with the membrane there is only confinement of the geotextile and no impregnation by soil particles. Besides, the lubrication of the membrane reduces friction with the geotextile, so the influence of friction on the test results is minimised. Independent on the confining material used the tensile stiffness is considerably increased in comparison to the result of the test in isolation.

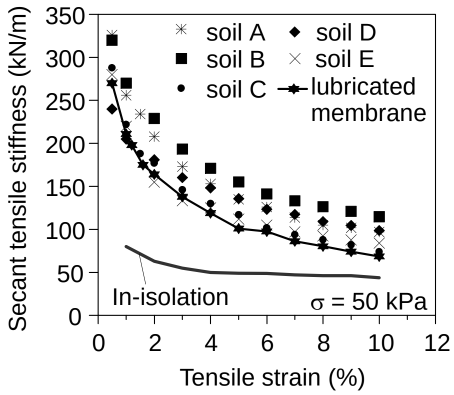


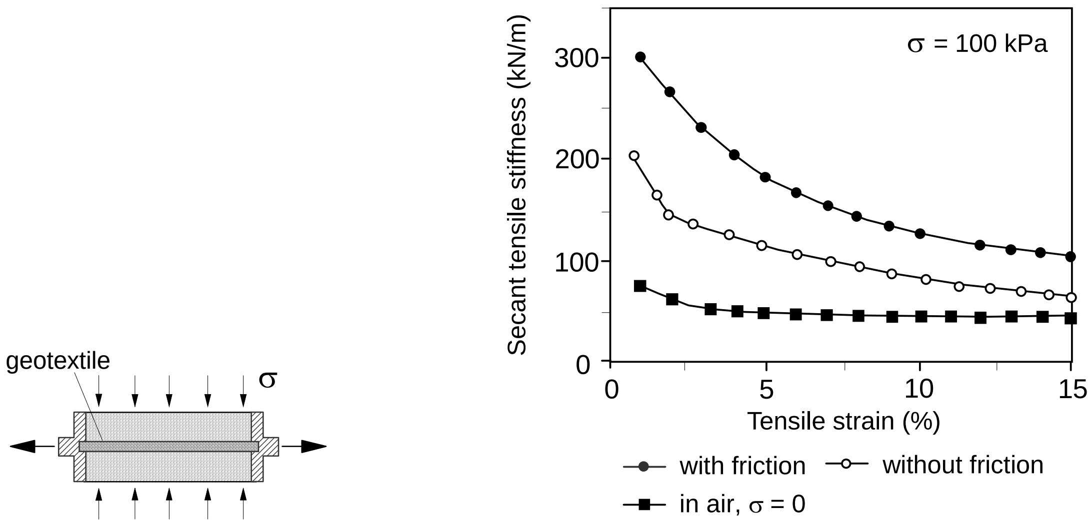
Fig. 34 Influence of confinement on the load-strain behaviour of a nonwoven geotextile (Palmeira et al. 1996).

In-soil tensile tests with different boundary conditions can be found in the literature. Different boundaries or test conditions are likely to yield to differences in the test results. Palmeira et al. (1996) have observed that soil arching is likely to occur in test arrangements such as that shown in Figure 33 due to the presence of the stiffer ends of the geotextile specimen and to the compressible central region of the geotextile layer that is effectively stretched. This effect is also likely to influence test results to an unknown extent. The values of tensile stiffness obtained under low strain levels (say, less than 1%) in in-soil tensile tests are also likely to be somehow inaccurate because of the influence of non uniform movements of clamps at the beginning of the test and due to accommodation or closure of small gaps between components of the apparatus.

An improved version of the apparatus with respect to arching effects is presented in Figure 35(a) (Boyle et al. 1996, Palmeira 1996), where the entire geotextile length is stretched. In this case, the side walls also work as clamps. In this arrangement, both soil arching and friction between soil and geotextile are avoided. Figure 35(b) shows that the secant tensile stiffness obtained for this arrangement is still significantly greater than that obtained with the geotextile in isolation, but smaller than the value obtained when friction between soil and geotextile specimen is allowed (Mendes et al. 2007).

The presence of soil particles inside the geotextile (Fig. 5) can increase its stiffness, as the presence of these particles further reduces the space available for fibre stretching, as schematically shown in Figure 36 (Mendes et al. 2007). This figure also shows results of tensile tests under confinement on a needle-punched polyester nonwoven geotextile (400 g/m^2) for different levels of impregnation of the geotextile by soil particles and a confining stress of

100kPa. Impregnation of the geotextile was achieved by spreading the soil on the geotextile surface and by vibrating the specimen to help particle intrusion. The level of impregnation (λ) was quantified as the ratio between the mass of soil particles per unit area and the mass geotextile fibres per unit area. In these tests confinement of the geotextile was provided by wooden plates with the face in contact with the geotextile specimen lubricated by layers of plastic films and grease. Figure 36 shows that the greater the mass of soil inside the geotextile the greater its tensile stiffness. However, a stiffer response of the geotextile due to impregnation will depend on the feasibility of soil particles intrusion in the fibre matrix and the amount of geotextile void space occupied by these particles. Mendes et al. (2007) have observed that the shape of the intruded soil particles also influenced the tensile stiffness increase, with round soil particles being less effective than angular particles for that matter.



(a) Frictionless in-soil tensile test (b) Comparison between test arrangements.

Figure 35. Results of in-soil tensile tests with and without friction between soil and geotextile (Mendes et al. 2007).

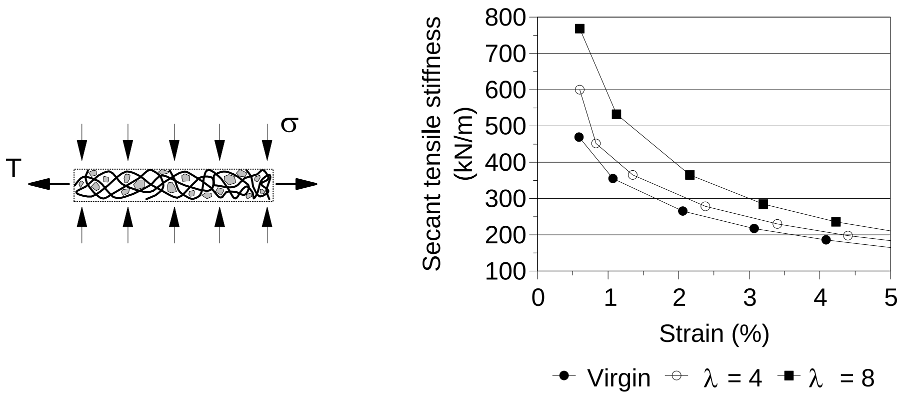
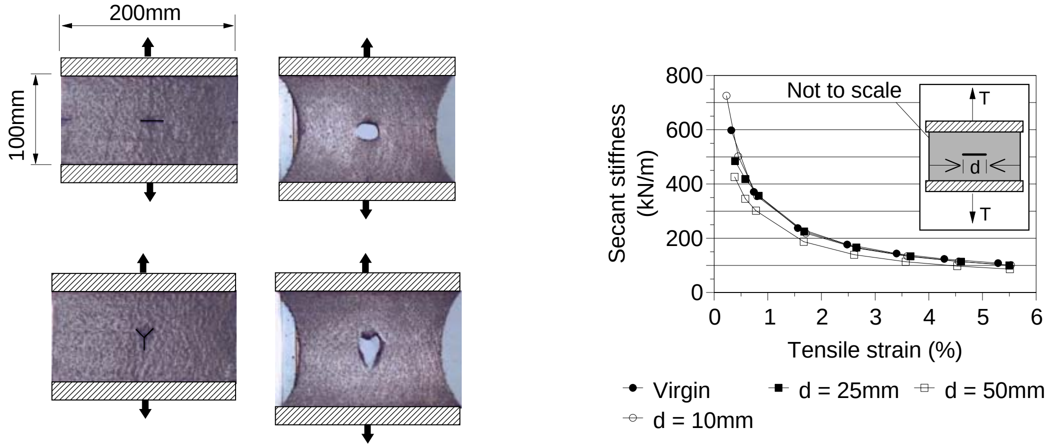


Figure 36. Influence of the impregnation of a nonwoven geotextile by soil particles on its tensile stiffness (Mendes et al. 2007).

The in-soil tensile test can also be useful to evaluate the influence of mechanical damages on the tensile properties of geotextiles. Figure 37(a) shows photographs of a horizontal cut and a "Y" shaped cut in geotextile specimens at the beginning of an in-air wide strip tensile test and for a tensile strain of 30%. It can be seen that the original horizontal cut evolves to an oval shape and the "Y" shaped cut evolves to a heart like shape. Figure 37(b) shows results of in-soil tensile tests on needle-punched nonwoven geotextile specimens (200g/m^2) with horizontal cuts of varying lengths. The soil used to confine the geotextile specimen was a uniform coarse sand and the confining stress used was equal to 100 kPa. Figure 37(b) shows that the results obtained for geotextile specimens with horizontal cuts with varying lengths are not much different from the result obtained for the intact and virgin geotextile specimen for tensile strains above 1%. Thus, confinement can reduce the detrimental effects of damages on the behaviour of nonwoven geotextiles, which may be relevant for applications of such materials as reinforcement, separators and, to some extent, filters.



(a) In-air wide strip tests on mechanically damaged geotextiles.

(b) Influence of confinement.

Figure 37. Influence of confinement on the tensile stiffness of mechanically damaged nonwoven geotextiles (Mendes et al. 2007).

Regarding in-soil tensile tests, in summary:

- The test is indicated for testing nonwoven geotextiles and can be useful for the investigation of the effects of damages on their mechanical behaviour.
- The boundaries can also affect the results and to isolate the effect of friction between soil and geotextile (lower bound test result) the original test arrangement proposed by McGown et al. (1982) with lubricated rubber membranes confining the geotextile or the arrangement where both soil and geotextile can deform laterally should be preferred.

RAMP TESTS

Ramp tests, or inclined plane tests, are relevant to the study of the stability of cover systems of waste disposal areas or for erosion control in slopes. In slopes of waste disposal areas, besides the cover soil, more than one type of geosynthetic can be present, such as geomembranes, geonets and geotextiles, to fulfil different roles, as shown in Figure 38. If the cover soil stability is not properly addressed, failure can occur. Figures 39(a) and (b) show examples of failures of cover soils on geosynthetic layers in slopes of waste disposal areas. The repair of such events is time consuming and expensive because usually a large fraction of the lining system must be repaired or substituted.

Figure 38. Geosynthetic layers in slopes of waste disposal areas.

The ramp test can be a useful tool to study and quantify the interaction between soils and geosynthetics in problems such as those shown in Figures 38 and 39, being one of its advantages the possibility of simulating the actual low stress levels on the interfaces caused by the relatively thin cover soil layer. The use of conventional direct shear apparatus for tests under low stress levels can result in misleading interface strength parameters (Girard et al. 1990, Giroud et al. 1990, Gourc et al. 1996). The ramp test basically consists of increasing the inclination of the ramp until a soil block slides on the geosynthetic layer fixed to the ramp surface, as shown in Figure 40. Similar test arrangements can be employed for tests on interfaces between different geosynthetics, such as geotextiles and geomembranes, for instance (Palmeira et al. 2002). Variations between equipment and testing conditions can be found in the literature (Girard et al. 1990; Giroud et al. 1990; Koutsourais et al. 1991; Girard et al. 1994; Gourc et al. 1996; Izgin and Wasti

1998; Lalarakotoson et al. 1999; Lopes et al. 2001; Wasti and Özdüzgün 2001, Palmeira et al. 2002, Palmeira and Viana 2003). Briaçon et al. (2002) describe a ramp test apparatus capable of performing tests under submerged conditions. The results obtained with such apparatus were validated by the authors in comparisons with results from large field experiments.

(a) Failure of cover soil (Dwyer et al. 2002).

(b) Failure of cover soil (Gross et al. 2002)

Figure 39. Failures of slopes of waste disposal areas.

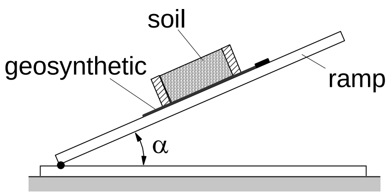


Figure 40. Ramp test.

If one examines the mechanics of the test (Fig. 41), assuming limit equilibrium conditions and a trapezoidal distribution of normal stresses along the soil-geosynthetic interface, the following equations for maximum and minimum normal stresses can be derived (Palmeira et al. 2002)

$$\frac{\sigma_{max}}{\sigma} = 4 - \frac{6x}{L} \qquad (1)$$

and

$$\frac{\sigma_{min}}{\sigma} = \frac{6x}{L} - 2 \qquad (2)$$

with

$$\frac{x}{L} = \frac{\cos[\alpha + \tan^{-1}(h/L)]}{2\cos\alpha}[1 + (\frac{h}{L})^2]^{1/2} \qquad (3)$$

and

$$\sigma = \frac{W\cos\alpha}{L} \tag{4}$$

where σ_{max} and σ_{min} are the maximum and minimum normal stresses on the interface at the edges of the soil block (Fig. 41), σ is the average normal stress on the interface, x is the distance between the lower soil block edge and the point of application of the normal force on the interface, α is the inclination of the ramp to the horizontal, h is the soil block height, L is the soil block length and W is the weight of the soil block.

Equations 1 to 4 suggest that the level of non uniformity of the distribution of normal stress along the soil-geosynthetic interface is a function of the dimensions of the soil block. The lower the ratio between soil sample height (h) and length (L), the more uniform is the distribution of normal stresses on the interface. This favours the use of long boxes with thin soil layers for testing.

The approach presented above can be considered a simple approximation to the actual stress conditions in the ramp test. To check the assumption of trapezoidal normal stress distribution along the interface, numerical analyses of ramp tests were performed using the computer code PLAXIS (Brinkgreve and Vermeer 1998). The soil properties relevant to this analysis were: soil unit weight equal to 17 kN/m^3, Young modulus of 20 MPa, Poisson ratio equal to 0.3 and friction angle equal to 35^o. Mohr-Coulomb failure criterion was used for the soil and the interfaces. Three lengths (0.5m, 2m and 10 m) of the soil block and different ramp inclinations were investigated.

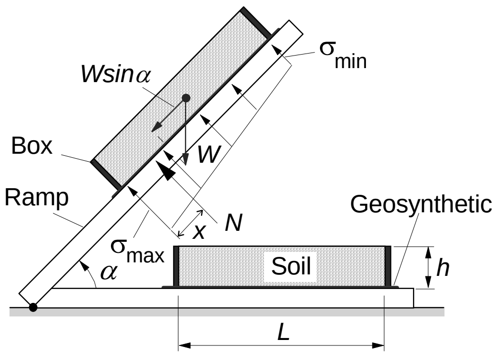


Figure 41. Equilibrium conditions in the ramp test (Palmeira et al. 2002).

The variations of normal stresses on the interface with its normalised length obtained by the numerical analysis, for a ramp inclination of 25^o, are presented in Figure 42 (Palmeira et al. 2002). The difference between maximum and minimum normal stress values increases considerably as the length of the block is reduced. For the 0.5 m long box the maximum normal stress was as much as 5 times the minimum normal stress. Non uniform normal stress distribution on the interface associated with the low stress level is a complicating factor for the analysis that should be avoided or minimised. The predictions of maximum and minimum normal stresses by equations 1 and 2 are also presented in Figure 42, which shows that the approach of a trapezoidal normal stress distribution on the interface was reasonably accurate for the conditions analysed.

Figure 42 also shows the effect of using a box with inclined sides, as suggested by Gourc et al. (1996), so as to these sides to coincide to the vertical direction at a ramp inclination of 25^o, for the 0.5 m long box. The results obtained by the numerical analyses show that a uniform stress distribution is obtained with this type of box, as expected, in contrast with the results obtained when the traditional box shape is used. These results indicate that for shorter boxes the configuration with inclined sides should be employed, although sample preparation may be a bit more complicate in this box. The results of the analytical and numerical analyses provide useful information for the design of this type of equipment.

The numerical analysis also allows the investigation of the evolution of force mobilisation in a geosynthetic layer and interface shear strength as the ramp inclination increases. Figures 43(a) and (b) show results from the numerical simulation of a ramp test (2m long and 0.23m high soil sample) on a sand-geotextile interface (Palmeira et al. 2002). For this analysis, the geotextile tensile stiffness was assumed equal to 70kN/m and the interface friction angle between soil and geotextile was assumed equal to 31^o. The geotextile had its top extremity anchored to the ramp. The friction angle between the geotextile and the ramp face was equal to 26^o. The sand layer had a Young modulus equal to 20MPa, Poisson ratio of 0.3, friction angle of 35^o and density equal to 17kN/m^3. Mohr-Coulomb failure criterion was used for the soil and values of shear stiffness for the soil-geotextile and geotextile-ramp interfaces were assumed equal to 3000 kN/m^3 and 25000 kN/m^3, respectively.

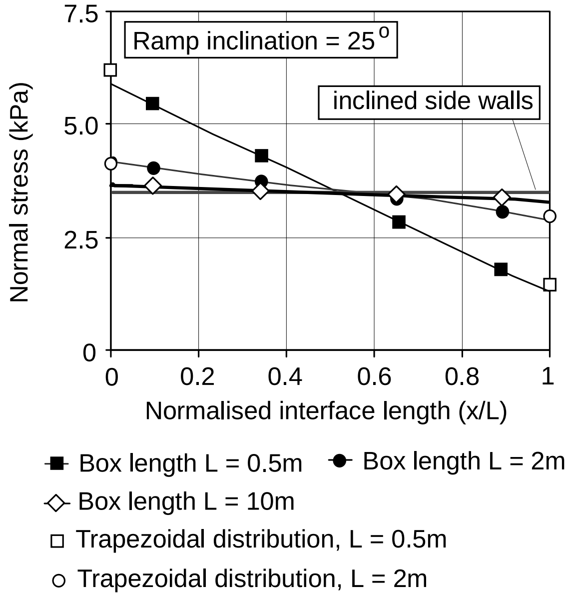
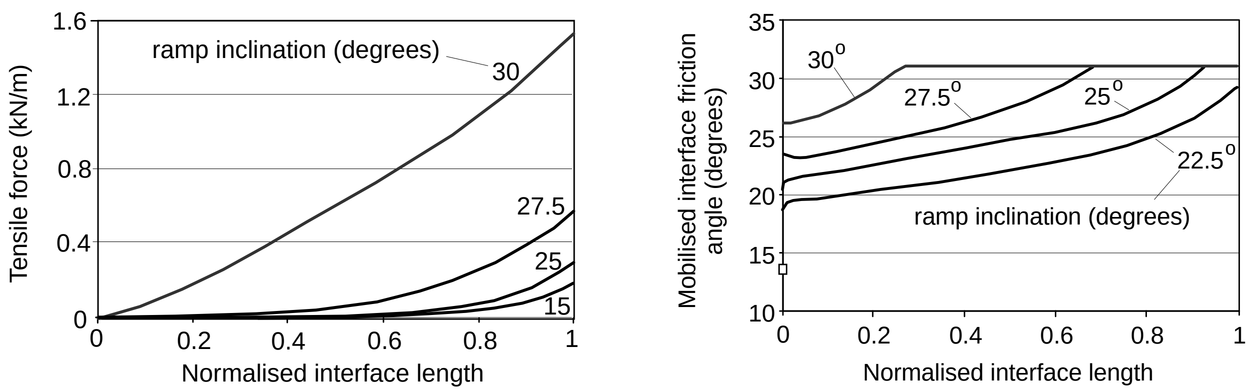


Figure 42. Numerical and analytical simulations of the ramp test (Palmeira et al. 2002).

Figure 43(a) shows a progressive mobilisation of tensile force on the geosynthetic as the ramp inclination increases. The entire geosynthetic length is tensioned only for ramp inclinations very close to the failure value (31^o). Mobilised interface friction angles versus normalised interface length are presented in Figure 43(b) and the progressive nature of the failure mechanism can also be clearly identified. A similar type of mechanism is likely to occur under field conditions, particularly for long slopes.



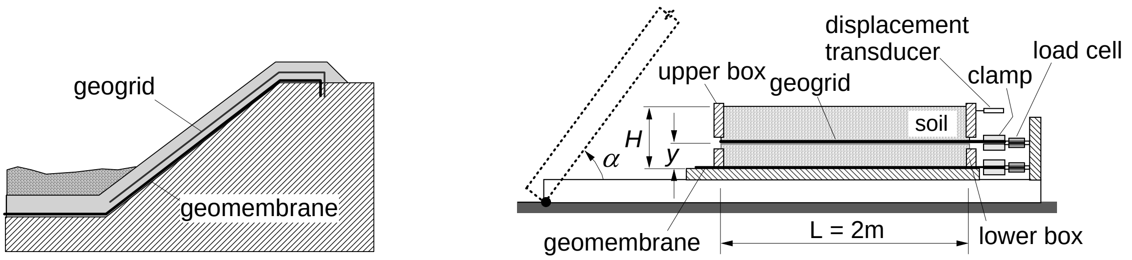
(a) Tensile force distribution along the geotextile length.

(b) Mobilised interface strength along the geotextile length.

Figure 43. Progressive mobilisations of tensile forces and interface strength in a ramp test (Palmeira et al. 2002).

Regarding field conditions, a possible solution to reduce tensile forces in a geomembrane in a slope of a waste disposal area is the use of reinforcement in the cover soil, as schematically shown in Figure 44(a). A large ramp test can be employed to investigate the potential benefit of such solution (Palmeira and Viana 2003). From the practical point of view, the easiest arrangement is to install the reinforcement directly on the geomembrane. However, this is not the most efficient position for the reinforcement. Figure 44(b) shows the test arrangement for a ramp test apparatus with 2m long boxes. The geomembrane below the soil and the geogrid buried in the soil are anchored to a rigid frame and load cells allow for the measurement of mobilised tensile loads during the test. The elevation of the geogrid can be varied to investigate its influence on the box displacements and on geosynthetics loads.

Figure 45 shows results of large scale ramp tests for the following cases: a cover soil on a geomembrane, cover soil reinforced by a geogrid installed at its mid-height and also a situation with the reinforced cover soil on a geomembrane protected by a geotextile (Palmeira and Viana 2003). The use of a geotextile on the geomembrane is a common measure to minimise damages to the geomembrane that might jeopardize its performance as a barrier. The geogrid was made of polyester with apertures 20mm x 20mm, thickness of 1.1mm and a tensile stiffness of 200kN/m. The geotextile, when present, was a nonwoven, needle-punched, geotextile made of polypropylene and with a mass per unit area equal to 200g/m^2. The geomembrane had a smooth surface and was made of high density polyethylene. The soil was a uniform coarse sand. Figure 45 shows the variation of box displacement versus ramp inclination. The presence of the geogrid reinforcement increased markedly the ramp inclination at failure from 26 degrees to 34 degrees. The arrangement with the geogrid and the geotextile increased the ramp inclination at failure a bit further and provided a less deformable system, with significant box displacements starting to occur only for ramp inclinations above 28 degrees. Thus, the presence of the reinforcement layer in the cover soil can allow higher slope inclinations or, for the same inclination of an unreinforced system, the presence of the reinforcement can provide additional safety against cover soil sliding on the geomembrane.



(a) Cover soil reinforcement. (b) Large ramp test apparatus.

Figure 44. Reinforcement of cover soils of waste disposal areas (Palmeira and Viana 2003).

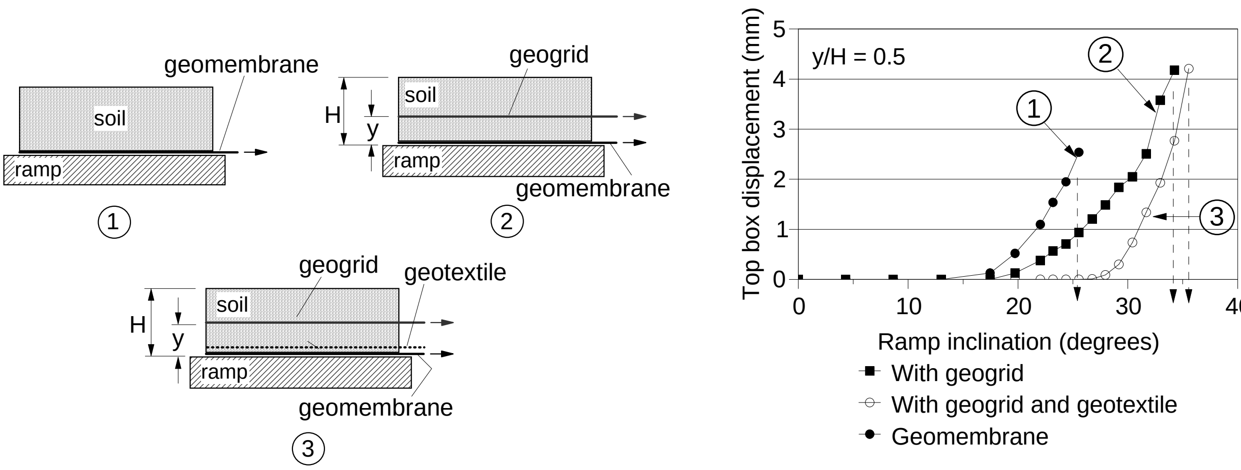


Figure 45. Effect of the presence of reinforcement on cover soil stability (Palmeira and Viana 2003).

Figure 46 presents the variation of ramp inclination at failure (α) for reinforced and unreinforced cover soils as a function of the position of the geogrid reinforcement (Palmeira and Viana 2003). The most efficient position for the reinforcement is at one third of the cover soil height, as might be expected from the analysis of stresses in the soil. The combination of geogrid in the soil and geotextile on the geomembrane provided the greatest beneficial effect to the system stability.

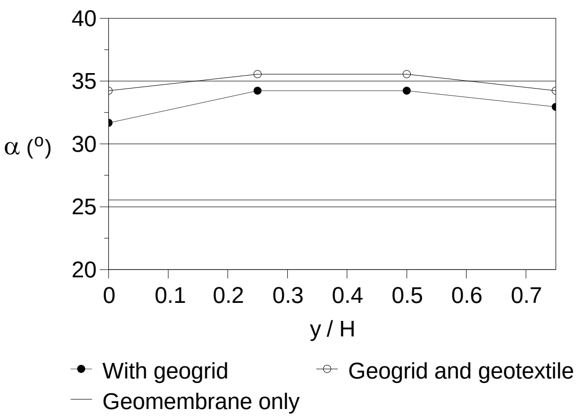


Figure 46. Influence of the reinforcement elevation on the ramp inclination at failure (Palmeira and Viana 2003).

The ramp test results also suggest that the tensile force mobilised in the geomembrane can be significantly reduced because of the presence of the reinforcement. Figure 47 shows the reduction on the force in the geomembrane due to the presence of the reinforcement in the cover soil at different values of normalised elevation. In all cases the ramp inclination was equal to the value at failure for the unreinforced case (α = 25.6^o in Fig. 47). The combination of geogrid and geotextile yielded the greatest reduction of tensile force in the geomembrane. The elevation of the geogrid also affected markedly that force, with the geogrid directly on the geomembrane being the least effective arrangement. However, it should not be ignored that the efficiency of the arrangement of the geogrid on the geomembrane also depends on the friction angle between both materials. Less friction between the geomembrane and the geogrid will certainly lead to a better performance of that arrangement.

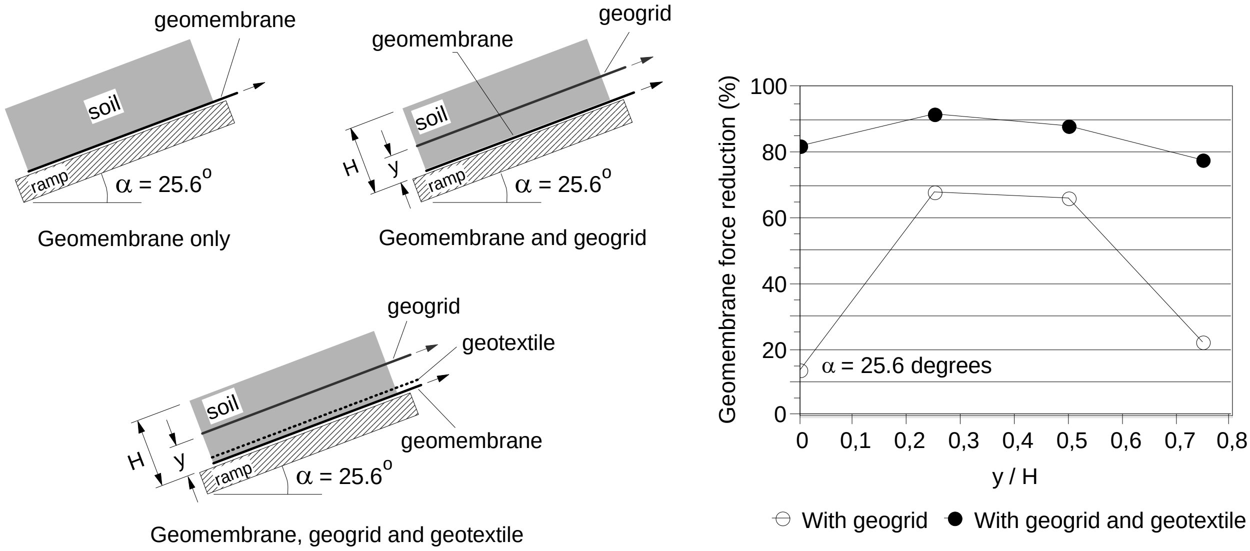


Figure 47. Reduction of mobilised geomembrane tensile forces against geogrid elevation.

The results presented above show the potentials of the use of the ramp test to investigate the behaviour of reinforced cover soils in waste disposal areas.

A summary of the conclusions on the ramp test is presented below:

- Ramp tests are appropriate for tests under low stress levels, which are typical in cover soils of slopes of waste disposal areas or in protection works against slope erosion.
- They are also useful for testing interface strength of multiple layers of geosynthetics or reinforced cover soils.
- They are easy to perform and long boxes or boxes with inclined lateral faces should be preferred.
- Long boxes with one of the geosynthetics ends anchored simulate more accurately the progressive failure nature of the interfaces in the region close to the slope crest. Nevertheless, depending on the dimensions of the test, it should be still considered as a rather rough model of the expected behaviour under field conditions.

CONCLUSIONS

This paper presented and discussed typical experimental and numerical methods for the study of soil-geosynthetic interaction. Emphasis was given to direct shear tests, pull-out tests, in-soil tensile tests and ramp tests. The main conclusions of this study are summarised below.

- Direct shear tests are simple tests to perform, but boundary conditions may influence the results of tests on reinforced soil samples, particularly for small shear boxes. The soil-geosynthetic shear interface, which is relevant for numerical analysis is difficult to evaluate, unless more sophisticated equipment and testing techniques are employed. The interpretation of tests with the reinforcement crossing the shear plane is difficult and extrapolations of results or conclusions obtained under such testing conditions to real structures should be done with caution.
- Pull-out tests results can be highly sensitive to boundary conditions. Lubrication of the wall internal frontal face or the use of a frontal sleeve are practical measures that minimise the influence of the box frontal face on the test results. The use of sleeve should be preferred as it may provide greater confidence on a significant reduction of the box frontal face influence, but theoretical and experimental data available suggests that further investigation is required to a better understanding on the influence of the box frontal wall on pull-out test results. For the usual large pull-out boxes dimensions, the sleeve length should not be smaller than 0.3 m. For low height boxes or long grids

compared to the sample height both sleeve and lubrication of the box frontal face should be used. Because other boundaries of the pull-out apparatus and soil sample dimensions may also influence test results, pull-out tests should be performed using large apparatus with samples height not less than 0.6m for the usual reinforcement lengths tested (typically less than 1m long). Accurate methods to predict the pull-out strength and response of geogrids for use in practice are yet to be developed, which enhances the importance of the pull-out test.

- In-soil tensile tests are useful for the study of the mechanical behaviour of geotextiles under confinement, particularly that of nonwovens. Factors such as impregnation of the geotextile by soil particles can increase even further the in-soil tensile stiffness of these materials, but this particular mechanism is still difficult to anticipate or predict under field conditions. In-soil tensile tests with the confinement of the geotexile imposed by lubricated rubber membranes can provide lower bound values of non woven geotextile tensile stiffness.
- Ramp tests are easy to perform and useful for the study of the stability of lining systems. They can also be employed to assess the effectiveness of cover soil reinforcement and the influence of water flow on the cover system stability conditions, for instance. Large testing equipment or boxes with inclined lateral faces should be preferred to minimise boundary effects and non uniform normal stress distributions on the soil-geosynthetic interface.
- Numerical analyses using the finite element method and the discrete element method can be useful tools for the understanding of the pull-out behaviour of geogrids and interaction mechanisms in other types of tests. They can also be employed to evaluate the limitations or the influence of boundary conditions on the test results. As numerical solutions are becoming more and more sophisticated and user friendly, their contributions to a better understanding on soil-geosynthetic interaction will certainly increase markedly in the coming years. However, users of computer codes must be sufficiently educated on numerical analysis and aware of modelling limitations and their implications on the accuracy of predictions.
- Despite how complex a testing device for the study of soil-geosynthetic interaction may be, it should be pointed out that the testing techniques available are, in most cases, still rather rough approximations of the behaviour of the geosynthetic in the field.
- The standardisation of testing devices and procedures for the evaluation of soil-geosynthetic interaction is of utmost importance in practical terms as well as for a faster improvement on the understanding on soil-geosynthetic interaction.

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