Effects of vegetation on the angle of internal friction of a moraine

Frank Graf1, Martin Frei2 and Albert Böll3

1 WSL Institute for Snow and Avalanche Research SLF, CH-7260 Davos Dorf, Switzerland. graf@slf.ch
2 Forest Department Grisons, CH-7000 Chur, Switzerland. martin.frei@afw.gr.ch
3 WSL Swiss Federal Institute for Forest, Snow and Landscape Research, CH-8903 Birmensdorf, Switzerland. albert.boell@wsl.ch

Abstract
Vegetation clearly affects soil strength, but how to take these effects into account in conventional calculations of slope stability is still an unsolved problem. To quantify the important influence of plant roots on shear strength of a moraine, we performed isotropic, consolidated-undrained triaxial compression tests with different confining pressures ($\sigma_3' = 50, 75, 100$ kPa). Three different types of samples were tested: A) planted soil and B) pure soil at low dry unit weight ($\gamma \approx 15.5$ kN/m$^3$) as well as C) pure compacted soil at $\gamma \approx 19$ kN/m$^3$. The planted samples were prepared with alder seeds ($Alnus incana$). For each sample type, failure lines ($k_f$ lines) were calculated using the peak shear strength points of the corresponding $p'$-$q$ paths. Robust statistics were performed to fit the failure lines and to test for significance.

No differences were found in the cohesion ($c'$) of the different soils. However, there was a significant difference in the angle of internal friction ($\Phi'$) of about 5° between the samples of pure soil at low dry unit weight and those of both compacted and planted soil. The vegetation effect is thus apparent as an increase in the angle of internal friction $\Phi'$ in planted soil compared to pure soil at the same dry unit weight. This finding can also be considered as a virtual increase in soil density (from $\gamma \approx 15.5$ to $\gamma \approx 19$ kN/m$^3$).

Keywords: triaxial compression tests, plants, slope stability, robust statistics

1 Introduction

During the last decade there has been a pronounced increase in the number of catastrophic events including shallow landslides and erosion processes after heavy rainstorms, particularly in mountainous regions, which has raised public awareness of the hazard (GÄRTNER 2003; RICKLI et al. 2004; BEZZOLA and HEGG 2007). Slope instability is thus a major concern for all those responsible for the protection of human lives and infrastructure against natural hazards. Consequently, methods to protect slopes against processes of erosion and sliding, and to stabilise those already affected are needed, as are reliable models and ways of estimating and predicting slope stability, and of calculating the factor of safety against failure.

For protecting slopes, joint technical and biological measures are particularly suitable as they combine environmental compatibility with cost effectiveness and durability (BÖLL et al. this issue). However, compared to purely technical constructions, these nowadays called eco-engineering methods face a major disadvantage. Until today, no very satisfactory ways of quantifying the biological effects on soil stability have been developed (BÖLL and GRAF 2001; CAZZUFFI and CRIPPA 2005). The contributions of technical constructions to soil strength can be directly fitted into conventional slope stability and security calculations, but this is not yet appropriately possible for biological measures.

The public has recently become much more aware of natural hazards in general, and demand for security has increased. Thus, precise information about the effects of plants on
slope stability is needed, and the development of methods to provide evidence for vegetation effects is an urgent objective. In particular, we need to find ways of including the influence of plants in conventional models for estimating the stability of natural slopes and embankments (JANBU 1954), which are generally based on the Mohr-Coulomb failure criterion ($\tau_f = c' + \sigma' \tan \phi'$, in LANG et al. 1996). For this purpose, the vegetation effects need to be assigned to shear strength, i.e. to the angle of internal friction $\Phi'$ or to the cohesion $c'$. 

To calculate and model vegetation effects on soil stability, suitable measurement techniques are necessary to properly address the relevant parameters. During the early history of soil mechanics, the direct shear test was the most popular approach, but it has some considerable disadvantages. To overcome some of its most serious limitations, the triaxial compression apparatus was developed in the 1930s (CASAGRANDE 1936; BARNES 1995). The triaxial compression test is more demanding and time consuming than the direct shear test, but it is also much more versatile. Several improvements have made it the appropriate choice, today, for experimental investigations of complex stress paths (LANG et al. 1996).

Nevertheless, direct shear tests are still very popular because they are simpler to apply and can be used to conduct experiments in the field. Accordingly, this method has been widely applied in investigations of soil reinforced by roots (WALDRON 1977; WU et al. 1979; STYCZEN and MORGAN 1995; WU and WATSON 1998; CAZZUFI and CRIPPA 2005), and tends to be the first approach used to quantify vegetation effects on soil stability. Even recently proposed approaches using fibre bundle models adapted for estimating the mechanical effects of vegetation on soil stability are finally based on direct shear tests (POLLEN and SIMON 2005; POLLEN 2007).

WU (1984) proposed, probably for methodical reasons, implementing the effects of plant roots on soil stability as an additive constant of the cohesion $c'$ in the Mohr-Coulomb failure criterion (Mohr-Coulomb: $\tau_f = c' + \sigma' \tan \phi'$; Wu: $sr = c' + \sigma' \tan \phi' + cr'$; with $s_r$ as the shear strength $\tau_f$ and $r$ for root). This approach has the advantages that, as a plain strength value, the additional cohesion $c_r'$ mobilised by roots, may be measured relatively simply by direct shear tests. Nevertheless, the soil stability conditions and, correspondingly, the effects of roots in the near-surface zone of the soil are not satisfactorily described by the cohesion $c'$. The stress-dependent term $\sigma' \tan \phi'$ represents the proper characteristics much better. During a simple shear test in which the only stresses measured are the normal and shear stresses on horizontal planes, it is not possible to define the stress state completely. As a result, the failure stresses measured may not represent the true strength of the soil. The results of laboratory tests on Cowden Till and on Blue London Clay (ATKINSON et al. 1991) showed that the strengths measured in simple direct shear tests differed from those measured in triaxial compression tests. The conventional interpretation of direct shear tests leads to a false cohesion intercept with friction angles smaller than those measured in triaxial compression tests (ATKINSON et al. 1991).

Recent experimental investigations on fibre reinforcement in sand yielded controversial findings, depending on the method applied. Using direct shear tests, YETIMOGLU and SALBAS (2003) found no improvement in the shear strength of the composite compared to pure sand, and OPERSTEIN and FRYDMAN (2000) reported an essentially constant angle of internal friction of soil reinforced by roots, but an increase in the apparent cohesion with increasing cross-sectional area and tensile strength of the roots. Accordingly, they interpreted the general increase in shear strength of the composite as the result of an increase in cohesion.

However, analyses based on triaxial compression tests revealed an increase in the angle of internal friction of a composite (fibre reinforced sand) compared to the untreated granular matrix (STAUFFER and HOLTZ 1995; CONSOLI et al. 2002). The addition of fibres to cohesion-less pure sand yielded an increase in the angle of internal friction without any change in the cohesion but when added to cemented sand (with cohesion), the increase in the angle of
internal friction went along with a decrease in cohesion (CONSOLI et al. 2002). Furthermore, it was found that the reinforcement effect generally correlates positively with the fibre aspect ratio, and, if the aspect ratio and concentration of fibres are kept constant, the composite strength is positively correlated with the length of the fibres (MICHALOWSKI and ČERMÁK 2003).

The triaxial compression test is better suited than the direct shear test at representing processes and characteristics of the superficial soil layers reasonably well. There is no rotation of the principal stresses, and, although stress concentrations still exist, they are significantly less. Normal stress is applied in three dimensions, and the area of shearing does not change during the test procedure. Furthermore, the failure plane can occur anywhere, and the stress paths can be controlled reasonably well. This means that complex stress paths in the field can be more effectively modelled in the laboratory. In particular, if undrained shear strength and the effective stress parameters of low-permeability material are needed, the triaxial compression test (consolidated undrained, with pore water pressure measurements) is by far more adequate (LAMBE and WHITMAN 1979; HOLTZ and KOVACS 1981; BARNES 1995; HEAD 1998).

The present study was undertaken to examine the influence of roots on the shear strength of moraine in a recent slide area. The focus is on the triaxial testing of moraine with a programme including pure soil and planted soil samples with low dry unit weight ($\gamma \approx 15.5$ kN/m$^3$) as well as on compacted pure soil samples with high dry unit weight ($\gamma \approx 19$ kN/m$^3$). Consolidated undrained triaxial testing was performed at different confining pressures to assess the effects of roots and dry unit weight on soil stability. The concept of “virtual density” (BÖLL and GRAF 2001) is proposed as a practical description of the shear strength of planted soils. “Virtual density” represents the higher dry unit weight of pure soil with a shear strength comparable to planted soil at lower dry unit weight.

2 Material and methods

2.1 Soil analysis and sample preparation

The soil investigated is a moraine of the subalpine landslide area “Schwandrübi” in Central Switzerland. The grain size distribution was analysed and the soil material classified (ASTM D422 2000, ASTM C136 2001, ASTM D2487 2002). Furthermore, the maximum dry unit weight at optimum water content, the liquid and plastic limits as well as the plasticity index were determined (ASTM D698 2000, ASTM D4318 2000). The coefficient of permeability was assessed with the oedometer method (LANG et al. 1996).

From oven-dried soil material (24 h at 105°C), the fractions ≤10 mm were used to prepare the soil samples for the triaxial compression tests. The soil material was moistened to a water content of 6% and tamped into PVC-plastic tubes (diameter: 70 mm; height: 140 mm). Subsequently, the specimens were dynamically compacted in three layers to the given dry unit weight and three different treatments were applied: A) soil ($\gamma \approx 15.5$ kN/m$^3$) planted with Alnus incana (L.) Moench, B) untreated soil with a dry unit weight of $\gamma \approx 15.5$ kN/m$^3$, and C) untreated compacted soil with a dry unit weight of $\gamma \approx 19$ kN/m$^3$. For treatment A) 15 alder seeds were applied to each sample and reduced to three seedlings after four weeks of growing. The samples were arranged completely randomly and maintained in a greenhouse for 20 weeks with 16 h of daylight daily and temperatures of 17°C during the day and 10°C at night. Totally, 15 samples were finally evaluated, with five from each treatment.

After the testing procedure, the cleaned plant roots were analyzed with a flatbed scanner and the root length was determined with the software WinRhizo® (2000). The root length per sample volume [cm/cm$^3$] was used as an indicator for plant growth.
2.2 Triaxial compression tests

Isotropic consolidated-undrained triaxial compression tests were performed to determine the shearing resistance of the soil samples (ASTM D 4767-95, 2000). Following FREI et al. (2003), the specimens were surrounded with a filter-paper drain and enclosed within a double latex membrane. The specimens were saturated with de-aired water under a pressure of 20 kPa. The initial degree of saturation was improved by applying a back pressure of 1200 kPa following the recommendations of BISHOP and HENKEL (1957). The B-value was checked according to DAY (2001). All samples had B-values greater than 0.98 and were assumed to be fully saturated. For each of the three treatments (A, B, C), compression tests were performed with effective confining pressures of $\sigma_3' = 50, 75, $ and 100 kPa with one or two specimens (Table 1). Axial stress, axial strain and pore water pressure were recorded during the shearing process. The axial strain rate was 0.015 % per minute. All tests were stopped once the loading piston had covered a distance of 20 mm.

Table 1. Confining pressures $\sigma_3'$ [kPa] and corresponding number of samples [n] tested for each of the three different treatments: A (soil planted with Alnus incana with $\gamma \approx 15.5 \text{ kN/m}^3$), B (untreated soil with $\gamma \approx 15.5 \text{ kN/m}^3$), and C (untreated compacted soil with $\gamma \approx 19 \text{ kN/m}^3$).

<table>
<thead>
<tr>
<th>Treatment</th>
<th>A</th>
<th>B</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>confining pressure $\sigma_3'$ [kPa]:</td>
<td>50 75 100</td>
<td>50 75 100</td>
<td>50 75 100</td>
</tr>
<tr>
<td>number of samples [n]:</td>
<td>2 1 2</td>
<td>2 1 2</td>
<td>2 1 2</td>
</tr>
<tr>
<td>dry unit weight $\gamma$ [kN/m$^3$]</td>
<td>15.3 15.6 18.9</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The shearing strength was determined according to the deviator stress ($\sigma_1 - \sigma_3$) at the yield-strength point in the stress-strain diagram for the highly compacted soil samples. This corresponds to the “peak” in the stress-strain diagram and to the local maximum in the $p'-q$ diagram (ZHU and ANDERSON 1998). For the planted and the pure soil samples at low dry unit weight, the stress-strain values were determined at “phase transformation”, which characterises more ultimate sample behaviour (NEGUSSEY et al. 1988). In the $p'-q$ diagram, this state is determined by the inflexion point of the path (ZHU and ANDERSON 1998; SPRINGMAN et al. 2003).

2.3 Statistical analysis

All statistical calculations were performed with the software R 2.5.0 (2007). For each of the treatments A, B, and C, failure lines (kf lines) were calculated, based on the peak shear strength points of the corresponding $p'-q$ paths, by performing all combinations of 3 out of 5 (C($5,3$)) without repetition. This resulted in 10 kf lines for each treatment (Equation 1).

$$C(n, k) = \binom{n}{k} = \frac{n!}{k!(n-k)!} = \frac{5!}{3!(5-3)!} = 10$$

where $C$ (binomial coefficient, also known as the “choose function”) is the number of all k-combinations from a set of n elements without repetitions, with $k = 3$ (confining pressure) and $n = 5$ (number of samples); see also Table 1.
To fit the failure lines, robust linear statistics were performed based on an MM-estimator (R-function: rlm) as proposed by Venables and Ripley (2002). Confidence intervals (1-\(\alpha\)) for the difference between 20% trimmed means (Wilcoxon 2005; function trimpb2) were calculated to compare the failure lines (intercept = cohesion \([c']\) and slope = angle of internal friction \([\Phi']\)) of the three treatments A, B, and C. The stress-strain data of the triaxial compression tests were subjected to local polynomial regression fitting (R-function: smooth.spline) after Hastie and Tibshirani (1990).

Additionally, robust Kruskal-Wallis rank sum tests (R-function: kruskal.test) and pairwise comparisons using Wilcoxon rank sum tests (R-function: pairwise.wilcox.test) were performed with correction for multiple testing according to Holm (1979).

In addition to the conventional mean values and standard deviations, a robust location parameter and measure of scale are given where appropriate, with the median and the median absolute deviation (mad), respectively.

3 Results

3.1 Soil and samples

The grain size distribution of the moraine, including the coarse parts up to 63 mm, is: 75.3% gravel, 15.5% sand, 7.1% silt, and 2.1% clay. The corresponding values of the soil material used for the triaxial compression tests (grain size \(\leq\)10 mm) are: 40.4% gravel, 37.1% sand, 17.0% silt, and 5.0% clay (Fig. 1). According to ASTM D2487 (2000), the moraine is classified as a clayey gravel with sand (GC-CL).

Fig. 1. Grain size distribution of the “Schwandrübi” soil material: ▲ = distribution curve of the moraine investigated, including the coarse grains up to 63 mm; ■ = distribution curve of the material used for triaxial compression tests, including grains \(\leq\)10 mm.
Further properties of the moraine include: the liquid limit (w_L: 21.5 %), plastic limit (w_P: 12.9 %), the plasticity index (I_P: 8.6 %), the specific gravity (D_s: 26.9 kN/m³), the maximum dry unit weight (γ: 21.9 kN/m³) with optimum water content (w: 7.9 %), and the coefficient of permeability (k: 6 x 10⁻¹⁰ m/s).

During the 20 weeks in the greenhouse, changes in the initial dry unit weight occurred. These were more pronounced in the planted soil (Δγ: 1.84 kN/m³ ± 0.67) and pure soil (Δγ: 1.86 kN/m³ ± 0.43) samples of low dry unit weight than in the compacted soil samples (Δγ: 0.6 kN/m³ ± 0.99), although the differences were not significant.

The comparison of the dry unit weights before and after the consolidation process of the three different treatments revealed significant differences in the values before consolidation (Kruskal-Wallis p-value: 0.0087) but not afterwards (Kruskal-Wallis p-value: 0.1111). The pair-wise comparisons of the dry unit weights before consolidation showed that those of the compacted soil samples were significantly higher than those of both the planted soil samples (p-value: 0.033) and the pure soil samples at low dry unit weight (p-value: 0.033) (Fig. 2, Table 2). The mean values and corresponding standard deviations of the three treatments were 15.26 kN/m³ ± 0.43 (A), 15.66 kN/m³ ± 0.64 (B), 18.85 kN/m³ ± 0.76 (C) before consolidation, and 19.98 kN/m³ ± 0.45 (A), 20.60 kN/m³ ± 0.75 (B), 20.70 kN/m³ ± 0.42 (C) after consolidation.

Within the plants in treatment A, the root length per soil volume [cm/cm³] varied considerably, with a minimum of 0.61 cm/cm³, a maximum of 2.17 cm/cm³, and a mean value of 1.26 cm/cm³ ± 0.71 (Table 2).

![Fig. 2. Dry volume weight γ [kN/m³] of the samples of planted, pure, and compacted soil before and after the consolidation process.](image-url)
3.2 Triaxial compression tests

The yielding at axial strain at failure $\varepsilon_f$ of the individual specimens in treatment A (planted soil) varied within the range of 2.3 % and 7.6 %, in treatment B (pure soil at low dry unit weight) between 2.4 % and 6 %, and in treatment C (compacted soil) within 1 % and 2.2 % (Table 2). No significant difference in $\varepsilon_f$ was detected between the treatments A and B (p-value: 1.0), but both have significantly higher values compared to treatment C (p-values: 0.024 each).

The deviator stresses ($q$) and, in good approximation, the mean effective stresses ($p'$) of the treatments A and B indicate a positive correlation with the confining pressure (Figs. 3, 4). This is, however, not the case for treatment C. There, the specimen tested at 75 kPa by far exceeds the others in both the mean effective stress ($p'$) and the deviator stress ($q$). Furthermore, one of the two specimens tested at 100 kPa has a lower $q$-value than one of those tested at 50 kPa (Table 2, Fig. 5).

Table 2. Properties of the individual samples used for the triaxial compression tests. $p_{ld}$ = planted soil samples (treatment A); $s_{ld}$ = pure soil samples (treatment B); $sh_{ld}$ = pure, compacted soil samples (treatment C); $V_0$ = volume of the samples after the greenhouse period; $\gamma_{bc}$ = dry unit weight before consolidation; $\gamma_{ac}$ = dry unit weight after consolidation; $\sigma_3$ = confining pressure; $p'$ = mean effective stress ($\sigma_1' + \sigma_3' / 2$); $q = \text{deviator stress } (\sigma_1' - \sigma_3' / 2)$; $\varepsilon_f = \text{strain at failure}$; root = root length per volume.

<table>
<thead>
<tr>
<th>sample</th>
<th>$V_0$</th>
<th>$\gamma_{bc}$</th>
<th>$\gamma_{ac}$</th>
<th>$\sigma_3$</th>
<th>$p'$</th>
<th>$q$</th>
<th>$\varepsilon_f$</th>
<th>root</th>
</tr>
</thead>
<tbody>
<tr>
<td>$p_{ld}$ 1</td>
<td>494</td>
<td>14.5</td>
<td>20.1</td>
<td>50</td>
<td>38.14</td>
<td>19.39</td>
<td>2.29</td>
<td>0.61</td>
</tr>
<tr>
<td>$p_{ld}$ 2</td>
<td>497</td>
<td>15.4</td>
<td>19.5</td>
<td>50</td>
<td>27.03</td>
<td>16.21</td>
<td>5.13</td>
<td>1.88</td>
</tr>
<tr>
<td>$p_{ld}$ 3</td>
<td>488</td>
<td>15.5</td>
<td>20.7</td>
<td>75</td>
<td>35.99</td>
<td>23.63</td>
<td>4.94</td>
<td>2.17</td>
</tr>
<tr>
<td>$p_{ld}$ 4</td>
<td>451</td>
<td>15.5</td>
<td>19.8</td>
<td>100</td>
<td>51.26</td>
<td>31.81</td>
<td>2.98</td>
<td>0.82</td>
</tr>
<tr>
<td>$p_{ld}$ 5</td>
<td>455</td>
<td>15.4</td>
<td>19.8</td>
<td>100</td>
<td>44.47</td>
<td>27.21</td>
<td>7.60</td>
<td>0.81</td>
</tr>
<tr>
<td>$s_{ld}$ 1</td>
<td>467</td>
<td>16.2</td>
<td>21.0</td>
<td>50</td>
<td>26.80</td>
<td>15.35</td>
<td>5.84</td>
<td>—</td>
</tr>
<tr>
<td>$s_{ld}$ 2</td>
<td>467</td>
<td>15.2</td>
<td>19.3</td>
<td>50</td>
<td>33.96</td>
<td>17.85</td>
<td>2.84</td>
<td>—</td>
</tr>
<tr>
<td>$s_{ld}$ 3</td>
<td>478</td>
<td>15.2</td>
<td>20.7</td>
<td>75</td>
<td>38.52</td>
<td>23.21</td>
<td>4.47</td>
<td>—</td>
</tr>
<tr>
<td>$s_{ld}$ 4</td>
<td>439</td>
<td>16.5</td>
<td>21.2</td>
<td>100</td>
<td>66.07</td>
<td>36.92</td>
<td>2.40</td>
<td>—</td>
</tr>
<tr>
<td>$s_{ld}$ 5</td>
<td>481</td>
<td>15.0</td>
<td>20.8</td>
<td>100</td>
<td>55.91</td>
<td>32.58</td>
<td>5.96</td>
<td>—</td>
</tr>
<tr>
<td>$sh_{ld}$ 1</td>
<td>528</td>
<td>19.1</td>
<td>20.5</td>
<td>50</td>
<td>41.87</td>
<td>25.22</td>
<td>2.20</td>
<td>—</td>
</tr>
<tr>
<td>$sh_{ld}$ 2</td>
<td>542</td>
<td>19.2</td>
<td>20.1</td>
<td>50</td>
<td>59.18</td>
<td>36.86</td>
<td>1.74</td>
<td>—</td>
</tr>
<tr>
<td>$sh_{ld}$ 3</td>
<td>537</td>
<td>19.4</td>
<td>20.9</td>
<td>75</td>
<td>90.72</td>
<td>58.25</td>
<td>1.75</td>
<td>—</td>
</tr>
<tr>
<td>$sh_{ld}$ 4</td>
<td>524</td>
<td>19.0</td>
<td>20.8</td>
<td>100</td>
<td>75.10</td>
<td>41.07</td>
<td>1.02</td>
<td>—</td>
</tr>
<tr>
<td>$sh_{ld}$ 5</td>
<td>429</td>
<td>17.5</td>
<td>21.2</td>
<td>100</td>
<td>62.97</td>
<td>30.82</td>
<td>1.07</td>
<td>—</td>
</tr>
</tbody>
</table>

The variance of the 10 failure lines ($k_f$ lines) of each treatment, represented by robust regression lines, is least in soil at low dry unit weight, slightly more in planted soil, and most in compacted soil (Figs. 3–5). Concerning the treatments A and B, the comparison of the trimmed means of each of the 10 transformed regression lines reveals a significant difference in the slope, i.e. in the angle of internal friction [$\Phi'$], and no significant difference in the intercept, i.e. in the cohesion [$c'$] (Table 3, Fig. 6).
Table 3. Comparison of intercept (cohesion \( c' \) [kPa]) and slope (angle of internal friction \( \Phi' \) [°]) of untreated soil at low dry unit weight (B) and planted soil (A), based on 1-\( \alpha \) confidence intervals for the difference between 20 % trimmed means (WILCOXON 2005); est. diff. = estimated difference.

<table>
<thead>
<tr>
<th>lower limit</th>
<th>upper limit</th>
<th>( \phi ) planted</th>
<th>( \phi ) untreated</th>
<th>est. diff.</th>
<th>( p )-value</th>
<th>dim.</th>
</tr>
</thead>
<tbody>
<tr>
<td>intercept ( [c'] ):</td>
<td>–2.4894</td>
<td>1.4011</td>
<td>–1.5635</td>
<td>–0.9269</td>
<td>–0.4503</td>
<td>0.6345</td>
</tr>
<tr>
<td>slope ( [\tan(\Phi')] ):</td>
<td>0.0146</td>
<td>0.0974</td>
<td>0.6766</td>
<td>0.6220</td>
<td>0.0583</td>
<td>\textbf{0.0076}</td>
</tr>
</tbody>
</table>

No significant differences were detected however, between the treatments A and C or between B and C neither in the slope \( [\tan(\Phi')] \) with a \( p \)-value of 0.9599 for A and C and one of 0.8607 for B and C nor in the intercept \( [c'] \) with a \( p \)-value of 0.6979 for A and C and one of 0.6997 for B and C. The range of dispersion of intercepts \( [c'] \) and slopes \( [\tan(\Phi')] \) was rather narrow for treatments A and B and remained within the same array but it was considerably wider for treatment C (Fig. 6).

The estimated angle of internal friction \( [\Phi'] \) and cohesion \( [c'] \) for the three treatments were 39.39° and –0.86 kPa for A, 34.25° and –0.04 kPa for B, and 40.14° and –3.89 kPa for C, respectively. The plants’ effect on the soil is reflected by an increase of 5° in the angle of internal friction, compared to pure soil samples at the same dry unit weight (\( \gamma \approx 15.5 \text{ kN/m}^3 \)), if we ignore the cohesion, which is virtually zero. Furthermore, the planted samples (A) at low dry unit weight (\( \gamma \approx 15.5 \text{ kN/m}^3 \)) mobilise the same angle of internal friction of about 40° as the compacted soil samples (C) at a dry unit weight of \( \gamma = 19 \text{ kN/m}^3 \) (Fig. 7).

---

Fig. 3. Untreated soil at \( \gamma \approx 15.5 \text{ kN/m}^3 \). To improve legibility, the stress-strain data are not shown completely. Black solid lines = stress-strain lines; dashed and solid blue curves = specimens tested at 50 kPa; solid red curve = specimen tested at 75 kPa; dashed and solid green curves = specimens tested at 100 kPa confining pressure; violet dotted lines = \( k_f \) lines; \( \triangle \) = inflexion points.
Fig. 4. Planted soil at $\gamma \approx 15.5$ kN/m$^3$. For legend, see Fig. 3.

Fig. 5. Untreated soil at $\gamma \approx 19$ kN/m$^3$. For legend, see Fig. 3.
Fig. 6. Comparison of intercept and slope values of the regression lines of each of the three treatments.

Fig. 7. Comparison of cohesion $c'$ [kPa] and angle of internal friction $\Phi'$ [°] of the three different treatments; planted, pure, and compacted soil. Dotted black = compacted soil ($\gamma \approx 19$ kN/m$^3$); dashed light grey = pure soil ($\gamma \approx 15.5$ kN/m$^3$); and solid dark grey = planted soil ($\gamma \approx 15.5$ kN/m$^3$). The dash-dotted black line represents the curve of the compacted soil displaced parallel to the intercept of the curve of the planted soil.
4 Discussion

The shear tests were performed with confining pressures ($\sigma_3'$) of 50, 75 and 100 kPa, representing the stress conditions in the superficial soil area, where root growth mainly occurs. Although even lower confining pressures would have been desirable, the value of 50 kPa marked the limit with respect to the practicability of shear tests given the laboratory equipment. However, the chosen conditions are well in accordance with those of other researchers investigating the strength of either soils reinforced by the roots of grass species (KIRSTEN and FOURIE 1999), or cemented and uncemented sand reinforced by fibres (CONSOLI et al. 2002; MICHALOWSKI and ČERMÁK 2003) and 3D inclusions (ZHANG et al. 2006).

The isotropic consolidated undrained triaxial compression tests clearly indicate an increase in the shear strength of the moraine investigated (clayey gravel with sand, GC-CL) with the presence of alder roots (Alnus incana). The gain in soil stability due to plant roots is exclusively based on the increase of the angle of internal friction $\Phi'$ from $34.3^\circ$ to $39.4^\circ$ without any noticeable change in apparent cohesion $c'$. The fact that some slight negative cohesion $c'$ was found may be attributed to some extent to the statistical evaluation procedure. This observation, however, is also consistent with the results of investigations of glacial till, where slightly positive or negative values were found (TIKA et al. 1996; IVERSON et al. 1998), and of fault gouge (MARONE et al. 1990; MARONE 1998; SCHOLZ 2002) and of the crater walls of volcanoes (MOON et al. 2005).

RATHBUN et al. (2008) conducted double direct shear experiments with different glacial till and obtained negative cohesion values as low as $c' = -6.8$ kPa, which is well below the values found in this study, namely $-0.86$ kPa for planted soil ($\gamma \approx 15.5$ kN/m$^3$), $-0.04$ kPa for untreated loose soil ($\gamma \approx 15.5$ kN/m$^3$), and $-3.89$ kPa for untreated compacted soil ($\gamma \approx 19.0$ kN/m$^3$). ATKINSON et al. (1990) conducted cylinder dispersion tests on a number of different soils to investigate true cohesion and dispersion in soils and found that reconstituted soils may exhibit positive, zero or negative cohesion. Moreover, for a given soil, the cohesion characteristics seemed to be governed by the chemistry of the pore water. Three characteristic behaviours of samples were distinguished, including non-dispersive with zero cohesion, non-dispersive with positive cohesion, and dispersive with negative cohesion. Comparable behaviours were found in soil aggregate stability tests (FREI et al. 2003; GRAF et al. 2006) conducted with the same moraine material used in the present study.

The soil-reinforcing effects of the plants are likely also to be attributable to the higher stability of unplanted soil with a higher dry unit weight, which in this case corresponded to an increase from $\gamma = 15.5$ kN/m$^3$ to $\gamma = 19$ kN/m$^3$.

Most investigations of the influence of vegetation on slope stability to date have depended on direct shear tests. The increase in shear strength because of the roots reported in all the studies, has normally been attributed to an increase in the apparent cohesion $c'$ (WALDRON 1977; SHEWBRIIDGE and SITAR 1989; WU and WATSON 1998; OPERSTEIN and FRYDMAN 2000; CAZZUFFI and CRIPPA 2005; WU 2007). An exception is the study of KIRSTEN and FOURIE (1999), who investigated the influence of three different grass species on the shear strength of frictional (pure) sand and clayey sand applying drained triaxial compression tests. In contrast to our results, they found not only an increase in the angle of internal friction $\Phi'$, but also on the apparent cohesion $c'$. The effect was more pronounced for the clayey sand than for the pure sand. However, there were some inconsistencies in their experiments. They added trace elements, fertiliser, organic compost, and vermiculite in the planted pure sand, but not in the clayey sand specimens and possibly not in the unreinforced control samples with pure sand. Consequently, the root growth in the pure sand was twice that in the clayey sand specimens. The mean values of the shear strength parameters were taken from experiments with different confining pressures, with $\sigma_1 = 100$ and 200 kPa for
both types of unreinforced soil, with $\sigma_1 = 20, 40, 100, \text{ and } 170 \text{ kPa}$ for the planted pure sand, and, with $\sigma_1 = 20, 80, \text{ and } 160 \text{ kPa}$ for the planted clayey sand. Furthermore, they applied a power function to describe the failure envelope of the unreinforced sand to allow for zero cohesion, as they expected no cohesion for this material, but used linear fits for the planted treatments. As a consequence, it is difficult to compare the effects of the different plants on soil stability both in their work and with the findings from our study.

Various investigations on peaty soil have been performed using triaxial compression tests, showing that the behaviour of the peaty soil is essentially frictional, with high angles of internal friction ($\Phi'$) and relatively small cohesion ($c'$) intercepts (Adams 1965; Edil and Dhowian 1981; Yamaguchi et al. 1985; Farrell and Hебиб 1998; Cola and Cortellazzo 2005). The high angles of internal friction are due to not entirely decomposed fibres intersecting the failure plane. This indicates that shearing resistance depends on the mutual orientation of fibres and failure plane. Undrained triaxial compression tests conducted by Yamaguchi et al. (1985) revealed significant differences in the angle of internal friction of samples taken vertically ($\Phi' = 52^\circ$) and horizontally ($\Phi' = 35^\circ$).

According to Wu et al. (1988), for roots to reinforce soil most effectively the vertical and horizontal growth should take up the stress applied.

From the methodical point of view, Farrell and Hебиб (1998) found that either the ring shear or direct shear test will most probably determine the shear resistance of the matrix, without the reinforcing effects of the fibres, a phenomenon already reported in Landva and La Rochelle (1983) who used a ring shear device to test Canadian Radforth peat. Possibly methodologically induced as well are the results of Yetimoglu and Salbas (2003) who analysed randomly distributed fibre-reinforced sands by applying direct shear tests, resulting in no discernible effect on shear strength, neither concerning the angle of internal friction $\Phi'$ nor the cohesion $c'$.

Findings from research on granular composites, reinforced by continuous filaments/fibres, result in similar method based conclusions (Morel and Gourc 1997; Consoli et al. 2002; Michalowski and Čermák 2003). Consequently, in recent years the triaxial compression test has become the method of choice as it yields a more promising description of fibre-reinforced soils than tests based on a “direct shear” concept (Sawicki 1983; De Buhan et al. 1989; Michalowski and Zhao 1993, 1996; Michalowski 1997; Consoli et al. 2002; Michalowski and Čermák 2002, 2003; Ghiaissian et al. 2004; Zhang et al. 2006; Tang et al. 2007). Investigations of a clayey soil reinforced with polypropylene fibre conducted by Tang et al. (2007) revealed an increase in both the angle of internal friction $\Phi'$ and the apparent cohesion $c'$. Similar findings were made by Chen and Loehr (2008) with Ottawa sand (Grade F-75). Research on silty sand and sandy soil in general (Park and Tan 2005) – our moraine material can be attributed to the latter – found that the inclusion of short fibres usually yields a distinct increase in the angle of internal friction, which is only sometimes accompanied by a change in the apparent cohesion (Michalowski and Čermák 2002, 2003; Zhang et al. 2006).

As found in the peat investigations, fibres orientated in the direction of maximum specimen extension contribute most to soil strength. In axisymmetric compression, as performed in our study, the maximum extension occurs in the horizontal plane. Conclusively, the contribution of the horizontal fibres is the largest (Michalowski and Čermák 2002). In our study, ductile sample deformation has been observed during triaxial compression tests, resulting in barrelling and, accordingly, in increasing specimen diameter (Head 1994). The apparent dependence of the angle of internal friction from the orientation of the reinforcing fibres indicates that in our study the root effect on the shear strength of soil is probably even underestimated as roots grew predominantly vertically, induced by the narrow diameter (70 mm) of the specimens. It further demonstrates that there may be pronounced discrep-
ancies between data gathered in the field and data obtained from laboratory experiments (GYSSELS et al. 2005).

The experimental tests of Michalowski (1997) showed that the angle of internal friction $\Phi'$ of the composite is larger than that of the granular matrix, whereas the applied model predicted changes only in the apparent cohesion $c'$. The reason for this is that the contribution of the filaments to the composite strength was included as a dissipation term independent of the composite stress state. Investigations of fine and coarse sand revealed that at small fibre concentrations (0.5 % Vol), the increase in the composite shear strength is larger for fine than for coarse sand. However, with increased fibre concentration, coarse sand benefits more than fine sand does.

The micromechanical behaviour of the fibre/matrix interface depends on the binding material properties in the soil, the normal stress around the fibre body, the effective contact area and the fibre surface roughness. The interface roughness is known to play an important role in reinforced soil systems (TANG et al. 2007). The strain in a coarse matrix with a high fibre concentration ($\approx 2 \%$) involves bending of the fibres to accommodate the change in the relative configuration of grains during the deformation process, which, in turn, enhances the interaction between grains and fibres. Furthermore, the reinforcing effect is positively correlated to the fibre aspect ratio given by the fibre length divided by its diameter, and, if this ratio is kept constant, with the length of the fibres. The reinforcement effect is more effective the larger the fibres are compared to the size of the grains (Michalowski and Čermáek 2003). Our findings from this study and GRAF et al. (2006) are similar, and we observed that shear strength and soil aggregate stability were positively correlated with the root length per soil volume.

In clayey soils, the “bridge” effect of fibres can efficiently impede the further development of tension cracks and deformation of the soil. Bond strength and friction at the interface seem to be the dominant mechanisms controlling the reinforcement benefit. In fibre-reinforced soil, interactions that occur at the interface between the fibre surface and the clay grains play key roles in the soil-mechanical behaviour. Furthermore, fibre-sliding resistance was observed to be strongly dependent on their surface roughness (FROST and HAN 1999; Tagnit-Hamou et al. 2005; TANG et al. 2007).

In our study of the moraine, we found, as in fibre-reinforced sand, that the planted specimens with the highest root length density (2.17 cm roots/cm$^3$ soil) mobilised the highest shear strength. That is, the angle of internal friction ($\Phi' = 41.03^\circ$, $c' = 0$) was 1.64° above the calculated mean value, which was derived from the 10 kf-lines of the planted specimens. The variance in the kf-lines of the planted specimens was higher than that of the pure soil at the same dry unit weight, which might be explained by the apparent differences in root length density. It is assumed that there exists a positive correlation between root length density and ramification with lateral roots. Therefore, a pronounced increase in the complexity of the roots’ 3D-structure is expected. Clearly, the higher complexity allows for an even more effective sand-fibre interaction and, consequently, further increases shear strength. In their studies on sand-reinforced with 3D inclusions, Zhang et al. (2006) found confirming results, i.e. a positive correlation between the shear strength and the complexity of the 3D structures added.

The reinforcing effects of plant roots on soil behaviour may be explained in a similar way to the interaction processes found between sand grains and artificial fibres. The results of our investigations of the effects of vegetation on the stability of a moraine fit quite well with those found in studies of soil reinforced with fibres. The isotropic consolidated undrained triaxial compression tests on moraine revealed a higher shear strength due to plant roots, reflected by a significant increase in the angle of internal friction $\Phi'$.

As discussed above, the angle of internal friction $\Phi'$ of planted soil was higher than that of pure soil at the same dry unit weight, and equal to the angle of internal friction of pure
soil samples at higher dry unit weight. This confirms the concept of the “virtual density” proposed by BÖLL and GRAF (2001). They recommend a new approach to quantify the biological effects on soil stability within eco-engineering measures, i.e. to quantify biological interventions on the stability of slopes affected by erosion and superficial sliding processes characterised by loose mixtures of grains with only small proportions of fine fractions and organic matter. A positive correlation is known to exist between the dry unit weight $\gamma$ and the angle of internal friction $\Phi'$ of pure soil. The “virtual density” approach simply explains the increase in soil stability due to vegetation as a virtual increase in the dry unit weight $\gamma$, and assigns the shear strength parameter $\Phi'$ of pure soil at the corresponding higher dry unit weight to the planted soil at lower dry unit weight. In this study, the planted soil with a dry unit weight of $\gamma \approx 15.5$ kN/m$^3$ may be regarded as unplanted compacted soil at $\gamma \approx 19$ kN/m$^3$. It is thus assigned the angle of internal friction of the unplanted compacted soil with $\Phi' = 40.1^\circ$, which is substantially the same as the effective value for the planted samples ($\Phi' = 39.4^\circ$).

The testing procedure we applied with triaxial compression tests as well as the robust statistical methods proved appropriate for assessing the effects of vegetation on soil stability and yielded comprehensible results. However, the operation sequence is time consuming and demanding, so the number of replications was rather low. Further tests are needed to verify our results, particularly with different soil types. As eco-engineering measures are not restricted to one plant species only, future investigations should quantify the simultaneous effects of different plant species on soil stability, including important symbiotic microorganisms that naturally occur together. Within such a community, the diversity of soil-reinforcing structures is considerably higher and, stabilisation processes are probably accelerated. Another step to cope better with natural conditions is to adapt the specimen size and/or their preparation procedure to allow more distinctive root growth in horizontal direction as well. Possible sources of data for this purpose are either undisturbed natural samples or installing large containers containing plants and taking samples by extracting drilling cores at appropriate size after the designated growing period.

Acknowledgements
We thank Prof. Sarah Springman for her challenging discussions on the meaning of the data and two anonymous reviewers for their helpful suggestions and comments as well as Silvia Dingwall for correcting the English. The PhD project of co-author Martin Frei, who provided the data, was funded by the Swiss Federal Office for the Environment FOEN.

5 References


Revised version accepted May 11, 2009