Shear strength of dry tills from the southern half of Norway in relation to bedrock geology

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Till is the dominant Quaternary sediment in Norway, where the till deposits mostly originate from local rocks with a transport distance less than five kilometres. Debris slides and flows are among the main geohazards on till-covered valley slopes, and in this context the shear strength of Norwegian tills is poorly studied. Therefore, a set of 33 near-surface, genetically independent till samples were collected from various locations in the southern half of Norway to investigate the shear strength and its relation to bedrock geology. The disturbed samples were sieved (<16 mm) and dried before they were tested in a large-scale direct shear box apparatus at normal stresses of 100, 200 and 300 kPa, and with a shear rate of 2.0 mm/min for approximately 40.0 mm horizontal displacement. Overall, the result regarding the average 'initial dry testing density' was 1.67 g/cm³, while the maximum shear stresses for the given normal stresses on average were 84.9 kPa, 163.2 kPa and 243.7 kPa, respectively. The average angle of friction and cohesion were 38.4° and 5.3 kPa. Furthermore, the samples were categorised into six regional rock provinces. The mapped rock type assumed to represent the origin of the till samples were based on their area of extent around the sample sites, and adjusted for by the late-Weichselian ice-flow directions. On average, the 'Precambrian basement' province locally affected by the Caledonian orogeny (mainly gneisses) had the highest angle of friction. In comparison, the 'Caledonian' province of overthrust sheets of sandstone and schist (mainly sandstones) had the lowest angle of friction. Although not conclusive, this study provides indications of a relationship between till shear strength and bedrock geology, suggesting that some provinces may, solely on the basis of their associated geological parameters, be more prone to debris slides and flows than others.

Keywords: till, shear box, shear strength, bedrock geology, landslide

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Introduction

The Norwegian bedrock geology is highly variable, both in terms of formation (igneous, metamorphic and sedimentary) as well as age, ranging from Precambrian to Permian (Johnsen, 1995). In addition to the Precambrian 'Basement', Caledonian rocks comprising overthrust sheets of both Precambrian rocks and sandstone and schist, metamorphic and igneous rocks, as well as Precambrian basement locally affected by the Caledonian orogeny, constitute the main part of the Norwegian bedrock (Geological Survey of Norway [NGU], 2016a). Regarding the formation of till, Boulton

(1974) stated that the processes of crushing, plucking and abrasion of rock masses depend largely on rock strength and hardness. The variation of the rocks also affects the composition of till, as it is normally composed of local rock material (Låg, 1948; Dreimanis et al., 1957). Due to the several ice ages (Vorren & Mangerud, 2006), till is regarded as the dominant Quaternary sediment in Norway (Bergersen & Garnes, 1972; Garnes, 1973; Haldorsen et al., 1983). For onshore areas, almost all Quaternary deposits are younger than 300 ka, and probably more than 90% of the present, remaining glacial deposits derive from the Weichselian glaciation, i.e., younger than 115 ka (Olsen et al., 2013).

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While the till cover in higher relief areas is mostly discontinuous and its thickness only locally exceeds two metres (Haldorsen & Krüger, 1990), a consistent layer of till is covering about 25% of the Norwegian mainland area (Thoresen, 2000; Olsen et al., 2013). Even though the transport distances of till may be several tens of kilometres (Clark, 1987), or even hundreds of kilometres (Dreimanis & Vagners, 1971), most of the Norwegian till material has been transported a relatively short distance, i.e., less than five kilometres (Reite, 1990; Thoresen, 2000). Studies done in the neighbouring Scandinavian countries, Sweden and Finland, have found similar results, concluding that the majority of tills are transported only a few kilometres (Perttunen, 1977).

In geotechnical aspects, till is generally regarded as a 'good' foundation soil (Milligan, 1976), although the shearing resistance is essential in the analysis of soil stability problems such as slope stability (Fredlund & Rahardjo, 1993; Allred, 2000; Das, 2010). When excluding floods, it is primarily slope stability issues, such as avalanches and landslides, which count for the main geohazards in Norway. Historically, landslides and avalanches in snow, rock and clay are the main causes of fatalities (Jaedicke et al., 2008), but even though debris slides and flows seldom result in human fatalities, they often cause damage to buildings and infrastructure. Valley slopes that are mostly covered in till are, consequently, most prone to such landslides (Bargel et al., 2011). Furthermore, due to the predicted future climate change, the frequency of debris slides and flows in Norway is expected to increase (Kronholm et al., 2007; Norem & Sandersen, 2014; Sandersen, 2014).

The Norwegian Water Resources and Energy Directorate (NVE) holds the overall responsibility for governmental management tasks regarding the prevention of accidents due to landslides (NVE, 2016). Additionally, the NVE, in cooperation with the Norwegian Public Roads Administration, the Norwegian Meteorological Institute, and the Norwegian National Rail Administration, regularly publish online national forecasts of debris-slide warnings (NVE, 2013). This warning system is based on the correlation between past landslide incidents and both meteorological and hydrological variables (Boje et al., 2014), thus not directly including parameters such as shear strength. Concerning the present and future potential of debris slides and flows in Norwegian valleys, as well as the geological limitations of the national warning system, it is of special interest for NVE to investigate the shear strength in a variety of tills, thereby resulting in the motivation for this project.

Regarding slope stability, the soil mass shear strength can be defined as *'the internal resistance per unit area that the soil mass can offer to resist failure and sliding along any plane inside it'* (Das, 2010, p. 365). Controlled shear tests have been a method to measure the shear strength of soils since the 1930s (Bishop & Eldin, 1950). However,

as also indicated by Thermann et al. (2006), it seems that homogeneous soils consisting of one or a few closely related fractions such as clay, as well as non-cohesive soils, such as sand, have been the most commonly used material for shear strength tests (e.g., Donald, 1956; Bjerrum & Landva, 1966; Palmeira, 1987; Yagiz, 2001; Miller & Hamid, 2007; Schnellmann et al., 2013). In comparison, it seems that fewer studies have been done on heterogeneous soils consisting of multiple fractions such as till or colluvium (e.g., Gan et al., 1988; Iverson et al., 1994; Vanapalli et al., 1996; Fannin et al., 2005; Thermann et al., 2006). Apart from limited and local studies such as construction project reports and master theses (e.g., Lund, 2013; Langåker, 2014; Langeland, 2016), the shear strength of Norwegian tills is rather poorly documented.

For cohesionless materials, the shear strength is usually expressed in terms of the so-called angle of internal friction. On this matter, the shear stress-deformation relations are largely influenced by the initial soil density. For densely packed soils, this is visualised by a significant peak in the shear stress-deformation curve due to dilatancy, before levelling out horizontally in an 'ultimate steady state condition'. In contrast, loose to medium-packed soils show either no peak or only a minor peak in the curve before reaching the ultimate, horizontal level (Kaniraj, 1988; Ishibashi & Hazarika, 2015). According to Simoni & Houlsby (2006), this critical (ultimate) shear strength represents the minimum shear strength that the soil can display in a shear test. Furthermore, geological parameters, such as particle size distribution and particle shape (Cornforth, 1973; Yagiz, 2001), as well as particle roughness and strength (Duncan et al., 2014), are known to have an effect on soil shear strength. Mineralogy is also known to influence soil shear strength (Fannin et al., 2005), and the critical state angle of shearing resistance is, according to Bolton (1986), principally a function of mineralogy. Therefore, since till largely originates from local rocks, differences in bedrock geology should to some extent also influence till shear strength, and, consequently, there should be some noticeable differences among different rock types and provinces.

Typically, the upper parts and surfaces of Norwegian till deposits are more porous than the underlying, consolidated till due to processes such as weathering and biological activity (Norem & Sandersen, 2014; Sandersen, 2014), which, in turn, may reduce the relative soil density (Dearman, 1991). Debris slides and flows are usually triggered in this upper and weathered part of the deposits (Sandersen, 2014). Rather than finding the maximum peak shear strength, which is more likely occurring in the underlying, relatively non-weathered and densely packed till, this study focuses on the critical shear strength of the upper, loose to medium-packed surface till.

Hence, this study presents results of the critical shear strength for 33 tills from various locations in the southern half of Norway, i.e., south of and including Nord-Trøndelag county (Fig. 1; Table 1 & 2). The study examines the test results with regard to the mapped bedrock geology, i.e., the samples are geographically divided into regional, major rock provinces and then further into rock types. This categorisation of samples into rock provinces is done for the purpose of displaying both internal differences and similarities, as well as to evaluate the relation between till shear strength and regional bedrock geology and the various associated rock types. Additionally, to distinguish between samples and provinces of presumably 'strong rocks' and 'weak rocks', brief descriptions of the typical compressive strengths of the dominating rock types of each province are included for an evaluation of this potential relation to the shear test results. In summary, the aim of the study is to investigate the potential

relation of till shear strength to bedrock geology, and evaluate if some provinces in the southern half of Norway may be more prone to debris slides and flows than others on the basis of bedrock geology and associated geological parameters. However, it must be emphasised that the aim of the study is not to replicate current individual field conditions regarding e.g., stress conditions, density and saturation for the sake of retrieving the actual, in situ shear strength, but to test different tills under equal test settings to show the differences and for the purpose of a comparative analysis. Due to the limited documentation of previous, comparable, Norwegian studies on till shear strength, the sampling and testing procedure are tailored for this study, and are therefore described in detail. Note that this is the first introductory article of the study, whereas a subsequent article will further investigate the samples and results with regard to geological parameters such as particle size distribution and particle shape.

Figure 1. Simplified bedrock geology map of the southern half of Norway illustrating the major rock provinces (named here as A–H) listed in Table 1, and the approximate locations of the 33 till samples listed in Table 2, modified from Norsk Betongforening [NB] (1988, as cited by Haugen & Lindgård, 2012, p. 3, fig. 22).

Table 1. A brief description of the major rock provinces (named here as A–H) in Norway, as illustrated on the map in Fig. 1, modified from NB (1988, as cited by Haugen & Lindgård, 2012) and NGU (2016a, b).

Rock province	Description
A	Extrusive and plutonic rocks in the Oslo region; mainly syenite, granite, monzonite and rhomb-porphry
B	Sedimentary rocks in the Oslo region; mainly slate, limestone and sandstone
C	Precambrian basement; mainly gneiss, granite, metamorphosed volcanic and sedimentary rocks
Caledonian rocks	
D	Overthrust sheets of Precambrian rocks; mainly metamorphosed plutonic rocks, particularly gabbro
E	Metamorphic and igneous rocks; mainly phyllite, mica schist, metamorphosed sandstone, gneiss and greenstone
F	Overthrust sheets of sandstone and schist; mainly sandstone, conglomerate and slate
G	Precambrian basement, locally affected by the Caledonian orogeny; mainly gneiss
H	Sedimentary rocks; mainly sandstone and conglomerate

Methods

Preliminary work and fieldwork

Due to the greater variety of bedrock geology in the southern half of Norway (NGU, 2016a), the northern half of Norway was not included in the study. When concentrating on the southern half, the study focused on collecting till samples from a variety of locations. Accordingly, the locations were based on a bedrock geology map from NGU (2016b), as well as a Quaternary geology map (NGU, 2016c) to ensure that the samples were collected from a variety of both geographical and geological provinces. Areas above the marine limit containing till deposits were identified and chosen for further investigation in the field. However, it is often considered difficult to distinguish between different types of till in the field without performing detailed investigations (Dreimanis, 1976; Haldorsen, 1982; Haldorsen & Krüger, 1990). Thus, for simplicity, the tills sampled in this study were independent of their genesis, i.e., no distinction was made between subglacial till and supraglacial till.

When in the field, the sampling of the tills was done manually with the use of a shovel and bucket. For practical reasons, the sample locations were chosen in natural or man-made slope cuts with minimal vegetation cover, preferably along side slopes to forest roads (Fig. 2A–C). The samples were collected from till deposits within roughly 1.5 m beneath the base of the organic top soil. This was done intentionally so that all samples were approximately from the same depth and therefore relatively similar concerning their possible exposure to processes such as weathering, regardless of the total till cover thickness. Furthermore, about 15 cm of the uppermost till cover was removed before sampling was done further into the deposit, thus preventing the mixing of other potentially non-local material such

as sand from winter road maintenance. Note that the samples collected are disturbed and do not represent the original in situ/field conditions. This was acceptable, as it is considered almost impossible to obtain undisturbed samples in this type of material (Andresen, 1979; Hencher, 2012). Likewise, due to the well-known range in size of solids forming a till, from clay size particles to boulders (Clarke, 1987; Bell, 2002), it was not practically possible to sample or examine the larger fractions such as cobbles and boulders in the laboratory. According to recommendations in BS 1377–7 (1990) one should not include particles larger than 20 mm in the shear box apparatus. Consequently, and by visual inspection, particles with a diameter clearly larger than 20 mm were manually sorted out on site and not included in the sampling. Organic material such as roots and insects were also as far as possible removed manually during the sampling. From NS–EN 932–1 (1996), the samples were weighed on site with a spring scale to follow the given standard mass recommendations for a sample with a maximum particle size of 20 mm, i.e., minimum 53 kg per sample. The collected samples were put in individual plastic barrels and sealed to prevent any mixing of the material during transport. For the same reason, both the shovel and the bucket were cleaned after each sampling.

Laboratory work

The laboratory work was performed at the Department of Geoscience and Petroleum at the Norwegian University of Science and Technology (NTNU–IGP). As it is known that the reproducibility and repeatability of direct shear test results may vary (Converse, 1953; Bareither et al., 2008a) due to factors such as human imprecision (Thermann et al., 2006), a major focus in this study was that all the samples were prepared and tested as identically as practically possible with the same procedure. This was done to exclude variables and

Table 2. Overview of the locations of 33 collected till samples from 29 municipalities in 11 counties in the southern half of Norway, also categorised by the rock provinces as shown in Fig. 1.

Sample no.	Location	Municipality	County	Rock province
1	Kvam	Nord-Fron	Oppland	E
2	Fosse	Sør-Fron	Oppland	F
3	Sustad	Lillehammer	Oppland	F
4	Raufoss	Vestre Toten	Oppland	C
5	Harestua	Lunner	Oppland	A/B
6	Røyne	Lier	Buskerud	A/B
7	Jondalen	Kongsberg	Buskerud	C
8	Brunkeberg	Kviteseid	Telemark	C
9	Valle	Valle	Aust-Agder	C
10	Heranostangen	Lom	Oppland	D
11	Leirvassbu	Lom	Oppland	D
12	Murane	Årdal	Sogn og Fjordane	D
13	Kaupanger	Sogndal	Sogn og Fjordane	D
14	Jordalen	Voss	Hordaland	D
15	Indre Oppedal	Gulen	Sogn og Fjordane	G
16	Vassenden	Jølster	Sogn og Fjordane	G
17	Tunga	Volda	Møre og Romsdal	G
18	Voll	Rauma	Møre og Romsdal	G
19	Meisalstranda	Neset	Møre og Romsdal	G
20	Skatval	Stjørdal	Nord-Trøndelag	E
21	Tømra	Selbu	Sør-Trøndelag	E
22	Korsvegen	Melhus	Sør-Trøndelag	E
23	Budalen	Midtre Gauldal	Sør-Trøndelag	E
24	Yset	Tynset	Hedmark	E
25	Åkerådalen	Rendalen	Hedmark	F
26	Steinbekkbua	Stor-Elvdal	Hedmark	F
27	Opphus	Stor-Elvdal	Hedmark	F
28	Flisa	Åsnes	Hedmark	C
29	Sørli	Kongsvinger	Hedmark	C
30	Sandsnessætra	Nannestad	Akershus	A/B
31	Hakadal	Nittedal	Akershus	A/B
32	Passebekk	Kongsberg	Buskerud	A/B
33	Enodden	Midtre Gauldal	Sør-Trøndelag	E

thereby increase the likelihood of obtaining an equal comparison basis, and, furthermore, so that the results could be linked to the geological parameters of the material. In addition to the main test, six samples were therefore retested for two complete tests to investigate the repeatability of the results.

Initially, the samples were sieved both mechanically and manually for a minimum of five minutes on one single 16 mm sieve in their original condition, thereby discarding larger particles. As previously described, BS 1377-7 (1990) recommends not including particles larger than 20 mm, so the 16 mm sieve was chosen due to

the likely presence of some larger, elongated particles in the samples. Based on NS-EN 932-2 (1999) and NS-EN 933-1 (2012), a small portion of material of minimum 2.6 kg from each sample was thereafter randomly selected by splitting with the use of a riffle box, this for the purpose of performing future studies such as particle size distribution. Then, about 40 kg of each sieved sample (<16 mm) were put in shallow pans and dried in an oven at $110^{\circ}\text{C} \pm 5^{\circ}\text{C}$ for a minimum of 16 hours, which according to ISO 17892-1 (2014) is normally enough time to achieve a completely dry material. The main reason for drying was to exclude the possibility that the moisture content could be a variable in the testing.

Figure 2. An example of a typical till sampling location, here from sample no. 14, Jordalen in Voss municipality, Hordaland county: (A) A non-vegetated slope cut reachable by car, (B) Overview of the sample site (pit marked by shovel and carpenter's ruler), (C) Excavation of sample pit (the carpenter's ruler is 1 m).

After this preparation procedure, the samples were ready to be tested in the SB2010 located at NTNU-IGP, which is a large-scale direct shear box apparatus from Testconsult Ltd. (Fig. 3A, B). The SB2010 is a modern, fully automated machine which incorporates a personal computer for operating the machine, for logging and displaying test data in real time and for reporting test results. During a shear test, the SB2010 automatically registers the shear stress, τ (rounded to whole numbers of kPa), several times per millimetre displacement. The lateral and vertical load capacity range are 0 to 100 kN with a maximum vertical load capacity of 1000 kPa with the use of a precision stepper motor and hydraulics, whereas applied loads are measured directly using calibrated load cells (0.1% FS). The maximum inner dimensions of the stainless steel sample box are 305 mm x 305 mm x 200 mm, i.e., length and width (fixed), and height (adjustable sample height), respectively. The SB2010 machine is in full accordance with BS 1377-7:1990 (Testconsult, 2012).

Before testing the individual samples in the SB2010, the sieved and dried material was firstly weighed in its storage container. The material was then carefully poured into the sample box with the use of a hand scoop (Fig. 4A). The pouring was done into the middle of the box, resulting in a randomised distribution within the

box from the middle and outwards. Note that both the surfaces and contact area of the sample box halves were free of lubrication such as grease. A total sample height of approximately 180 mm was used, and as recommended in BS 1377-7 (1990) the sample was divided into three equally vertical-sized layers. These layers were manually divided and prepared with the use of two self-made wooden tools, ensuring that each layer in every sample was horizontally levelled at best possible equal vertical height, regardless of individual operator accuracy (Fig. 4B). Partly based on BS 1377-7 (1990), each layer was also compacted by doing five drops with a 4 kg weight (kettlebell) from a fixed height of approximately 20 cm on top of a steel plate covering the entire layer (Fig. 4C). According to recommendations in the newer ASTM D3080/D3080M (2011) the layer was, after compaction, 'scarified' before establishing a new layer. This scarification was performed with a garden hand fork on the first two layers, i.e., three times in two perpendicular directions over the entire layer area, to avoid distinct layer segregations (Fig. 4D). The last layer was compacted before performing a visual inspection of nine fixed points, i.e., the four corners, sides and the middle, measuring with a ruler the distance in millimetres from top edge of the box down to the top compacted surface of the sample (Fig. 4E, F). This was done to calculate the volume of material contained in

Figure 3. An overview of (A) the SB2010 large-scale direct shear box apparatus located at NTNU-IGP, and (B) with the installed, empty sample box.

Figure 4. The figures show the shear box testing procedure by preparing the sample for shearing: (A) Pouring of sieved (<16 mm) and dried till material into the middle of the sample box, (B) Levelling of a total of three equally vertical sized layers (approximately 60 mm each) with self-made wooden tools, (C) Initial compaction of each layer by doing five drops from a fixed height of approximately 20 cm with a 4 kg kettlebell on top of a steel plate, (D) Scarifying the two first compacted layers three times in two perpendicular directions with a garden hand fork to avoid distinct layering, (E) Filled shear box before final compaction, (F) Visual measurement of the sample height from nine fixed points (corners, sides and the middle) after compaction for calculation of sample volume and thereafter the initial dry testing density.

the sample box. Finally, the steel plate cover was put on top of the compacted material and the sample box was installed in the shear box apparatus. Most of the excess spilled material was with best effort collected and put back into the storage container before it was weighed again, hereby giving the weight of the material contained in the sample box. With approximate information of both mass and volume, the 'initial dry testing density', ρ_d , of the sample was calculated once per sample on the first test.

Even though the SB2010 has a programmable shear rate up to 10 mm/min (Testconsult, 2012), it seems from other shear test studies that they, in general, have used considerably lower rates, e.g., 0.5 mm/min (Fannin et al., 2005), 0.01 mm/min (Gan et al., 1988) and 0.005 mm/min (Miller & Hamid, 2007). In addition, ISO 17892-10 (2004) recommends a rate not higher than 0.5 mm/min. However, the purpose of shearing the sample slowly is, according to ASTM D3080/D3080M (2011), to allow pore-water pressures to dissipate. The samples in this study were, as described, completely dry, thereby avoiding the potential build-up of pore-water pressure. Furthermore, since the water content may also have an influence on the shear strength due to the force of suction (Rahardjo et al., 1995; Fredlund et al., 1996; Lommler, 2012), drying the samples excluded this variable as well. Therefore, the samples were tested with a higher shear rate of 2.0 mm/min.

Regarding shear distance, ASTM D3080/D3080M (2011) recommends that the sample should be sheared to at least 10% relative lateral displacement, i.e., minimum 30.5 mm for the SB2010. Although the SB2010 can shear the sample up to a distance of 50 mm (Testconsult, 2012), this may lead to spillage of material outside the sample box halves, as well as to a significantly decreased shearing area. Thus, it was decided to shear the samples for a horizontal distance of approximately 40.0 mm, which was also similar to the shear distance in the study done by Bareither et al. (2008b).

As recommended by both BS 1377-7 (1990) and ASTM D3080/D3080M (2011), three shear tests with different normal stress, σ , were performed on all 33 samples. The three chosen levels of applied normal stress were 100, 200 and 300 kPa, as in the study by Skuodis & Tamošiūnas (2014). These levels of normal stress are relatively high when compared with other studies such as Fannin et al. (2005) and they do not reflect current field conditions. However, the main reason for selecting these levels of normal stress was former laboratory experience at NTNU-IGP showing that the SB2010 was somewhat inaccurate regarding constant loading during tests with a normal stress below 50 kPa (G. Vistnes, pers. comm., 2015). Before each shear test the samples were vertically preloaded at 350 kPa for a period of three minutes, thereby ensuring that all samples had an equal starting condition.

After completion of each shear test, the sample box was emptied, cleaned and refilled as described. On this matter, it must be noted that the samples were reused,

which is not recommended in both BS 1377-7 (1990) and ASTM D3080/D3080M (2011), probably due to the potential alteration or deterioration of the material when shearing, e.g., particle crushing. However, as Norwegian rocks are generally recognised as strong (Palmstrøm, 1997; Grimstad et al., 2007), the degree of potential particle crushing was considered to be low. Additionally, the sample box was not exposed to vibration, which would result in an increased particle rearrangement into a denser state. The samples were therefore still in a loose packed state when shear tested. As mentioned, it is known that an increase in the density increases the peak shearing resistance (Simoni & Houlsby, 2006; Smith, 2014). Hence, shearing the samples in a loose packed state will minimise the risk of alteration or deterioration since there should be little or no dilatancy (Donald, 1956). Thus, the reuse of samples was regarded as having an insignificant influence on the geological parameters and the results. This reuse of the samples was also why lubrication of the sample box was avoided, as it would mix with the dry material thereby reducing the dryness. After completing the three shear tests, both the sample box and the shear box apparatus, as well as the other equipment, were thoroughly cleaned to avoid any mixing of material between different samples.

Supplementary work

From the locations (Table 2), a simplified bedrock geology map of the southern half of Norway was used as a basis for categorising the sample sites into regional, major rock provinces (Fig. 1; Table 1). For verification, the sample locations by GPS-coordinates were compared to the bedrock geology map from NGU (2016b). In addition to the transport distance of till, another key aspect in this context was the influence of the ice-flow directions which, during the different phases of the Weichselian glaciation, changed due to the shifts of the ice divide (Vorren, 1977). Both regional and local studies by e.g., Bergersen & Garnes (1971, 1972) and Reite (1994) have shown that the Weichselian ice-flow directions have altered quite significantly. Thus, for simplicity, only the major ice-flow directions on a national scale in the late-Weichselian were considered when evaluating the character of the possible rock material assumed to constitute the till samples.

After categorising the sample sites with respect to the major rock provinces A–H retrieved from the bedrock geology map, seven out of eight provinces were represented. Province H, which by far was also the smallest province, was not represented. Similarly, province B was the second smallest in size, but it also overlapped with province A. Due to the relatively small area of province B, as well as its overlap with province A, there was a possibility of mixing of the rock types between these two provinces. However, due to the main ice-flow directions in this southeastern part of Norway

Figure 5. (A) Reconstructed ice-sheet flow regime of the late-Weichselian in the southern half of Norway, where red lines indicate ice-flow directions from the ice divide towards the coastline, modified from Ottesen et al. (2005, p. 1048, fig. 13). (B) Bedrock geology map showing the rock type(s) outside and within a full circle with a radius of five kilometres from sample no. 3, Lillehammer (prov. F), categorised as inside/near the ice divide, (C) Bedrock geology map showing the rock type(s) outside and within a half-circle with a radius of five kilometres from sample no. 16, Jølster (prov. G). This sample is categorised as outside the ice divide with a westward ice-flow direction illustrated by the black arrow. (B) and (C) are modified from NGU (2016b).

(Fig. 5A), it was considered more likely that province A was influenced by province B, rather than the opposite. For simplicity, provinces A and B were therefore combined as one province, noted as A/B. Hence, between five to seven samples were collected from each of the remaining six provinces A/B–G (Table 2).

When combining the bedrock geology map (NGU, 2016b) with the sample sites and the late-Weichselian ice-flow directions (Fig. 5A), a full (360°) or half (180°) circle with a radius of about five kilometres was drawn around the center of the sample sites and used as outer limits for sorting out the most likely rock type or types constituting the till samples. A full circle was used for the samples considered to be in or near the area of the ice divide, since the ice-flow directions were unspecified in these areas (Fig. 5B). For samples considered to be outside the ice divide, a half circle was made from the sample site ‘downstream’ of the major ice-flow direction. Thus, for an ice-flow direction towards the west, rock types east of the sample site were included, while rock types farther west of the sample site were excluded, as this would be ‘countercurrent’ and therefore considered unlikely (Fig. 5C). After excluding areas on the map where the bedrock is covered, i.e., rivers, lakes and fjords, the up to three most dominant rock types within the remaining radius zone were selected and listed for each sample site. Their assumed influence regarding till content was based on their individual area size within the half or full circle, which were approximately measured in the planar view and calculated. The first rock type listed represents

the rock type in the area from which the sample itself was collected, independent of its area size, whereas the remaining rock types are listed with a chronological and decreasing respective percentage share.

The maximum shear stress registered by the SB2010 for the whole displacement length for each shear test was further processed to estimate the angle of friction and cohesion by linear regression analysis, restraining the fit of the data to cohesion ≥ 0 kPa. According to BS 1377–7 (1990), the reported angle of friction, ϕ , was rounded to the nearest 0.5°, while the cohesion, c , was rounded to one decimal place of kPa. The samples and associated results were then summarised in tables for each of the six major rock provinces, including e.g., the average and the standard deviation (SD). In addition, the obtained results were processed for the purpose of graphical visualisation of shear stress vs. horizontal displacement. This simplified plotting was done by choosing the registered shear stress value closest to each whole millimetre from the entire displacement length of each shear test. When processing all the samples in each province, both the highest and lowest registered values (independent of sample number), as well as the average for all samples, for each ‘whole millimetre’ were plotted. This made it possible to visualise the average curve, and that all the individual curves for each province thus lie somewhere within the minimum and maximum curves.

Results

When considering all the 33 samples, the average initial dry testing density, ρ_d , was 1.67 g/cm³ (SD = 0.15 g/cm³; median = 1.68 g/cm³). The lowest and highest registered values of ρ_d were 1.39 g/cm³ (sample no. 5, prov. A/B) and 2.04 g/cm³ (no. 11, prov. D), respectively, i.e., a difference of 0.65 g/cm³. For the main shear test with 100 kPa applied normal stress, the average shear stress value, τ , based on the maximum value for each sample, was 84.9 kPa (SD = 3.0 kPa; median = 85.0 kPa), with 78.0 kPa (no. 30, prov. A/B) as the lowest value and 91.0 kPa (no. 12, prov. D) as the highest value. With the increase of the normal stress to 200 kPa, the average shear stress was 163.2 kPa (SD = 6.5 kPa; median = 162.0 kPa), with 152.0 kPa (no. 30, prov. A/B) and 177.0 kPa (no. 16, prov. G) as the lowest and highest values, respectively. For 300 kPa normal stress, the average shear stress was 243.7 kPa (SD = 9.6 kPa; median = 246.0 kPa), with 229.0 kPa (no. 11, prov. D, and no. 30, prov. A/B) as the lowest value and 269.0 kPa (no. 12, prov. D) as the highest registered value. Thus, the differences in the shear stress results were up to 13.0 kPa, 25.0 kPa and 40.0 kPa for 100, 200 and 300 kPa applied normal stress, respectively. Furthermore, the diversities in shear stress resulted in a similar variety of estimated angles of friction, ϕ , and cohesion, c . The average angle of friction was 38.4° (SD = 1.3°; median = 38.5°), spanning from 36.0° (no. 26, prov. F) as the lowest, to 41.5° (no. 12, prov. D) as the highest registered value, i.e., a difference of 5.5°. Cohesion ranged from 0.0 kPa (nos. 12 and 14, prov. D, and nos. 20 and 21, prov. E) to 11.3 kPa (no. 22, prov. E), with an average of 5.3 kPa (SD = 3.5 kPa; median 5.3 kPa).

Regarding the mapped bedrock geology, the five samples in province A/B ('Extrusive and plutonic rocks/Sedimentary rocks in the Oslo region'), except for sample

no. 6, were dominated by syenites and granites. Apart from sample no. 8, the other five samples in province C ('Precambrian basement') were dominated by a variety of gneisses and granites. Province D ('Caledonian rocks, Overthrust sheets of Precambrian rocks') was generally dominated by various mangerite, gabbro, amphibolite and gneiss, whereas sample no. 13 was solely represented by anorthosite. Province E ('Caledonian rocks, Metamorphic and igneous rocks') was the most variable regarding the number of represented rock types in the samples. Overall, mica schist, mica gneiss, amphibolite, greenstone, metasandstone and phyllite were heavily represented in province E. Furthermore, province F ('Caledonian rocks, Overthrust sheets of sandstone and schist') was relatively homogeneous regarding rock types, as the samples were dominated by sandstone. Lastly, province G ('Caledonian rocks, Precambrian basement, locally affected by the Caledonian orogeny') was also relatively homogeneous regarding rock types, as all of the five samples were dominated by various gneisses and migmatites. The results regarding both bedrock geology and shear test are further elaborated by the individual rock provinces (Figs. 6 & 7; Tables 3–14) and then summarised in Table 15.

Considering the six samples retested for the purpose of repeatability assessment (Table 16), the results were variably equal to, higher and lower than the ones obtained from the main test. Analysing all six samples for the three test series performed, i.e., a total of nine shear tests per sample, the SD for the shear stress results for 100, 200 and 300 kPa normal stress ranged between 1.5–4.0 kPa, 1.7–4.7 kPa, and 2.3–8.1 kPa, respectively. For the angle of friction, the SD varied between 0.3° and 1.3°, while the SD for the cohesion ranged between 2.1 and 7.5 kPa.

Table 3. Rock types within 5 km distance for the five sample sites from rock province A/B, extrusive and plutonic rocks / sedimentary rocks in the Oslo region, from NGU (2016b).

Rock province	Sample no.	Rock types
A/B	5	Syenite, quartz syenite (~86%); Granite, granodiorite (~14%)
	6	Phyllite, mica schist (~28%); Rhomb-porphry (~27%); Granite, granodiorite (~16%); Other (~29%)
	30	Syenite, quartz syenite (~68%); Granite, granodiorite (~32%)
	31	Syenite, quartz syenite (~83%); Unspecified volcanic rocks (~8%); Granite, granodiorite (~7%); Other (~2%)
	32	Granite, granodiorite (~67%); Monzonite, quartz monzonite (~24%); Rhomb-porphry (~9%)

Table 4. The shear test results for the five till samples from rock province A/B.

Rock province	Sample no.	Initial dry testing density, ρ_d (g/cm ³)	Max. shear stress, τ , with normal stress, σ (kPa)			Angle of friction, ϕ (°)	Cohesion, c (kPa)
			$\sigma = 100$ (kPa)	$\sigma = 200$ (kPa)	$\sigma = 300$ (kPa)		
A/B	5	1.39	89.0	158.0	248.0	38.5	6.0
	6	1.65	85.0	159.0	235.0	37.0	9.7
	30	1.76	78.0	152.0	229.0	37.0	2.0
	31	1.59	85.0	171.0	248.0	39.0	5.0
	32	1.62	87.0	167.0	246.0	38.5	7.7
Average		1.60	84.8	161.4	241.2	38.0	6.1
Standard deviation		0.13	4.1	7.6	8.7	0.9	2.9
Median		1.62	85.0	159.0	246.0	38.5	6.0
Maximum		1.76	89.0	171.0	248.0	39.0	9.7
Minimum		1.39	78.0	152.0	229.0	37.0	2.0
Difference		0.37	11.0	19.0	19.0	2.0	7.7

Table 5. Rock types within 5 km distance for the six sample sites from rock province C, Precambrian basement, from NGU (2016b).

Rock province	Sample no.	Rock types
C	4	Augen gneiss, granite, foliated granite (~18%); Amphibolite, hornblende gneiss, mica gneiss (locally migmatitic) (~62%); Sandstone, slate (~14%); Other (~6%)
	7	Augen gneiss, granite, foliated granite (~84%); Dioritic to granitic gneiss, migmatite (~8%); Amphibolite, hornblende gneiss, mica gneiss (locally migmatitic) (~4%); Other (~4%)
	8	Rhyolite, rhyodacite, dacite, keratophyre (~34%); Quartzite (~40%); Basalt (~26%)
	9	Granite, granodiorite (~88%); Amphibolite, hornblende gneiss, mica gneiss (locally migmatitic) (~7%); Augen gneiss, granite, foliated granite (~4%); Other (~1%)
	28	Gabbro, amphibolite (~24%); Dioritic to granitic gneiss, migmatite (~43%); Augen gneiss, granite, foliated granite (~33%)
	29	Augen gneiss, granite, foliated granite (~3%); Dioritic to granitic gneiss, migmatite (~81%); Gabbro, amphibolite (~16%)

Table 6. The shear test results for the six till samples from rock province C.

Rock province	Sample no.	Initial dry testing density, ρ_d (g/cm ³)	Max. shear stress, τ , with normal stress, σ (kPa)			Angle of friction, ϕ (°)	Cohesion, c (kPa)
			$\sigma = 100$ (kPa)	$\sigma = 200$ (kPa)	$\sigma = 300$ (kPa)		
C	4	1.62	88.0	172.0	247.0	38.5	10.0
	7	1.64	87.0	172.0	256.0	40.0	2.7
	8	1.76	85.0	164.0	248.0	39.0	2.7
	9	1.73	87.0	173.0	259.0	40.5	1.0
	28	1.82	82.0	158.0	231.0	36.5	8.0
	29	1.81	82.0	158.0	235.0	37.5	5.3
Average		1.73	85.2	166.2	246.0	38.7	5.0
Standard deviation		0.08	2.6	7.1	11.1	1.5	3.5
Median		1.75	86.0	168.0	247.5	38.8	4.0
Maximum		1.82	88.0	173.0	259.0	40.5	10.0
Minimum		1.62	82.0	158.0	231.0	36.5	1.0
Difference		0.20	6.0	15.0	28.0	4.0	9.0

Table 7. Rock types within 5 km distance for the five sample sites from rock province D, Caledonian rocks, overthrust sheets of Precambrian rocks, from NGU (2016b).

Rock province	Sample no.	Rock types
D	10	Mangerite to gabbro, gneiss and amphibolite (~96%); Olivine rock, pyroxenite (~3%); Phyllite, mica schist (~1%)
	11	Mangerite to gabbro, gneiss and amphibolite (~100%)
	12	Mangerite to gabbro, gneiss and amphibolite (~74%); Gabbro, amphibolite (~16%); Dioritic to granitic gneiss (~10%)
	13	Anorthosite (~100%)
	14	Mangerite-syenite (~84%); Anorthosite (~12%); Dioritic to migmatitic gneiss (~3%); Other (~1%)

Table 8. The shear test results for the five till samples from rock province D.

Rock province	Sample no.	Initial dry testing density, ρ_d (g/cm ³)	Max. shear stress, τ , with normal stress, σ (kPa)			Angle of friction, ϕ (°)	Cohesion, c (kPa)
			$\sigma = 100$ (kPa)	$\sigma = 200$ (kPa)	$\sigma = 300$ (kPa)		
D	10	1.89	85.0	160.0	242.0	38.0	5.3
	11	2.04	81.0	159.0	229.0	36.5	8.3
	12	1.79	91.0	170.0	269.0	41.5	0.0
	13	1.68	84.0	165.0	246.0	39.0	3.0
	14	1.71	84.0	162.0	251.0	39.5	0.0
Average		1.82	85.0	163.2	247.4	38.9	3.3
Standard deviation		0.15	3.7	4.4	14.6	1.9	3.6
Median		1.79	84.0	162.0	246.0	39.0	3.0
Maximum		2.04	91.0	170.0	269.0	41.5	8.3
Minimum		1.68	81.0	159.0	229.0	36.5	0.0
Difference		0.36	10.0	11.0	40.0	5.0	8.3

Table 9. Rock types within 5 km distance for the seven sample sites from rock province E, Caledonian rocks, metamorphic and igneous rocks, from NGU (2016b).

Rock province	Sample no.	Rock types
E	1	Phyllite, mica schist (~47%); Metasandstone, mica schist (~31%); Amphibolite, hornblende gneiss, mica gneiss (locally migmatitic) (~18%); Other (~4%)
	20	Greenstone, amphibolite (~54%); Slate, sandstone, limestone (~38%); Conglomerate, sedimentary breccia (~7%); Other (~1%)
	21	Phyllite, mica schist (~77%); Quartzite (~17%); Greenstone, amphibolite (~6%)
	22	Greenstone, amphibolite (~29%); Slate, sandstone, limestone (~37%); Unspecified volcanic rocks (~21%); Other (~13%)
	23	Greenstone, amphibolite (~5%); Mica gneiss, mica schist, metasandstone, amphibolite (~65%); Quartzite (~30%)
	24	Mica gneiss, mica schist, meta-sandstone, amphibolite (~97%); Greenstone, amphibolite (~3%)
	33	Mica gneiss, mica schist, metasandstone, amphibolite (~92%); Quartzite (~6%); Greenstone, amphibolite (~2%)

Table 10. The shear test results for the five till samples from rock province E.

Rock province	Sample no.	Initial dry testing density, ρ_d (g/cm ³)	Max. shear stress, τ , with normal stress, σ (kPa)			Angle of friction, ϕ (°)	Cohesion, c (kPa)
			$\sigma = 100$ (kPa)	$\sigma = 200$ (kPa)	$\sigma = 300$ (kPa)		
E	1	1.70	88.0	171.0	247.0	38.5	9.7
	20	1.67	82.0	163.0	250.0	39.5	0.0
	21	1.41	82.0	163.0	246.0	39.5	0.0
	22	1.54	89.0	164.0	243.0	37.5	11.3
	23	1.55	83.0	157.0	232.0	36.5	8.3
	24	1.44	87.0	158.0	240.0	37.5	8.7
	33	1.53	80.0	155.0	233.0	37.5	3.0
Average		1.55	84.4	161.6	241.6	38.1	5.9
Standard deviation		0.11	3.5	5.4	6.9	1.1	4.7
Median		1.54	83.0	163.0	243.0	37.5	8.3
Maximum		1.70	89.0	171.0	250.0	39.5	11.3
Minimum		1.41	80.0	155.0	232.0	36.5	0.0
Difference		0.29	9.0	16.0	18.0	3.0	11.3

Table 11. Rock types within 5 km distance for the five sample sites from rock province F, Caledonian rocks, overthrust sheets of sandstone and schist, from NGU (2016b).

Rock province	Sample no.	Rock types
F	2	Sandstone (~65%); Metasandstone, mica schist (~22%); Quartzite (~6%); Other (~7%)
	3	Sandstone (~100%)
	25	Sandstone (~100%)
	26	Sandstone (~88%); Quartzite (~10%); Conglomerate, sedimentary breccia (~1%); Other (~1%)
	27	Sandstone (~93%); Conglomerate, sedimentary breccia (~6%); Limestone, dolomite (~1%)

Table 12. The shear test results for the five till samples from rock province F.

Rock province	Sample no.	Initial dry testing density, ρ_d (g/cm ³)	Max. shear stress, τ , with normal stress, σ (kPa)			Angle of friction, ϕ (°)	Cohesion, c (kPa)
			$\sigma = 100$ (kPa)	$\sigma = 200$ (kPa)	$\sigma = 300$ (kPa)		
F	2	1.59	89.0	167.0	249.0	38.5	8.3
	3	1.43	81.0	156.0	232.0	37.0	5.3
	25	1.73	85.0	159.0	237.0	37.0	8.3
	26	1.90	84.0	157.0	230.0	36.0	11.0
	27	1.46	82.0	158.0	240.0	38.5	2.0
Average		1.62	84.2	159.4	237.6	37.4	7.0
Standard deviation		0.20	3.1	4.4	7.5	1.1	3.4
Median		1.59	84.0	158.0	237.0	37.0	8.3
Maximum		1.90	89.0	167.0	249.0	38.5	11.0
Minimum		1.43	81.0	156.0	230.0	36.0	2.0
Difference		0.47	8.0	11.0	19.0	2.5	9.0

Table 13. Rock types within 5 km distance for the five sample sites from rock province G, Caledonian rocks, Precambrian basement, locally affected by the Caledonian orogeny, from NGU (2016b).

Rock province	Sample no.	Rock types
G	15	Dioritic to granitic gneiss, migmatite (~97%); Augen gneiss, granite, foliated gneiss (~3%)
	16	Dioritic to granitic gneiss, migmatite (~100%)
	17	Dioritic to granitic gneiss, migmatite (~99%); Olivine rock, pyroxenite (~1%)
	18	Dioritic to granitic gneiss, migmatite (~100%)
	19	Dioritic to granitic gneiss, migmatite (~92%); Amphibolite and mica schist (~8%)

Table 14. The shear test results for the five till samples from rock province G.

Rock province	Sample no.	Initial dry testing density, ρ_d (g/cm ³)	Max. shear stress, τ , with normal stress, σ (kPa)			Angle of friction, ϕ (°)	Cohesion, c (kPa)
			$\sigma = 100$ (kPa)	$\sigma = 200$ (kPa)	$\sigma = 300$ (kPa)		
G	15	1.74	89.0	174.0	255.0	39.5	6.7
	16	1.62	86.0	177.0	253.0	40.0	5.0
	17	1.69	86.0	170.0	253.0	40.0	2.7
	18	1.71	84.0	161.0	239.0	38.0	6.3
	19	1.75	84.0	157.0	244.0	38.5	1.7
Average		1.70	85.8	167.8	248.8	39.2	4.5
Standard deviation		0.05	2.0	8.5	6.9	0.9	2.2
Median		1.71	86.0	170.0	253.0	39.5	5.0
Maximum		1.75	89.0	177.0	255.0	40.0	6.7
Minimum		1.62	84.0	157.0	239.0	38.0	1.7
Difference		0.13	5.0	20.0	16.0	2.0	5.0

Table 15. A summary of the results of the average and the individual highest and lowest values based on all samples regarding initial dry testing density, ρ_d , shear stress, named here as τ_1 - τ_3 (for normal stress 100, 200, and 300 kPa, respectively), angle of friction, ϕ , and cohesion, c , sorted into the distinct rock provinces A/B-G with their corresponding values in parenthesis.

Average ρ_p , high to low (g/cm ³)		Average τ_1 , high to low (kPa)		Average τ_2 , high to low (kPa)		Average τ_3 , high to low (kPa)		Average ϕ , high to low (°)		Average c , high to low (kPa)	
D (1.82)		G (85.8)		G (167.8)		G (248.8)		G (39.2)		F (7.0)	
C (1.73)		C (85.2)		C (166.2)		D (247.4)		D (38.9)		A/B (6.1)	
G (1.70)		D (85.0)		D (163.2)		C (246.0)		C (38.7)		E (5.9)	
F (1.62)		A/B (84.8)		E (161.6)		E (241.6)		E (38.1)		C (5.0)	
A/B (1.60)		E (84.4)		A/B (161.4)		A/B (241.2)		A/B (38.0)		G (4.5)	
E (1.55)		F (84.2)		F (159.4)		F (237.6)		F (37.4)		D (3.3)	
High to low, ρ_d Low to high, ρ_d High to low, τ_1 Low to high, τ_1 High to low, τ_2 Low to high, τ_2 High to low, τ_3 Low to high, τ_3 High to low, ϕ Low to high, ϕ High to low, c Low to high, c											
(g/cm ³)		(kPa)		(kPa)		(kPa)		(°)		(kPa)	
D (2.04)	A/B (1.39)	D (91.0)	A/B (78.0)	G (177.0)	A/B (152.0)	D (269.0)	A/B (229.0)	D (41.5)	F (36.0)	E (11.3)	D (0.0)
F (1.90)	E (1.41)	A/B (89.0)	E (80.0)	C (173.0)	E (155.0)	C (259.0)	D (229.0)	C (40.5)	C (36.5)	F (11.0)	E (0.0)
C (1.82)	F (1.43)	E (89.0)	D (81.0)	A/B (171.0)	F (156.0)	G (255.0)	F (230.0)	G (40.0)	D (36.5)	C (10.0)	C (1.0)
A/B (1.76)	C (1.62)	F (89.0)	F (81.0)	E (171.0)	G (157.0)	E (250.0)	C (231.0)	E (39.5)	E (36.5)	A/B (9.7)	G (1.7)
G (1.75)	G (1.62)	G (89.0)	C (82.0)	D (170.0)	C (158.0)	F (249.0)	E (232.0)	A/B (39.0)	A/B (37.0)	D (8.3)	A/B (2.0)
E (1.70)	D (1.68)	C (88.0)	G (84.0)	F (167.0)	D (159.0)	A/B (248.0)	G (239.0)	F (38.5)	G (38.0)	G (6.7)	F (2.0)

Table 16. A summary of the results for two additional shear tests performed on six of the 33 till samples.

Sample no.	Test 2: Max. shear stress, τ , with normal stress, σ (kPa)			Test 3: Max. shear stress, τ , with normal stress, σ (kPa)		
	$\sigma = 100$ (kPa)	$\sigma = 200$ (kPa)	$\sigma = 300$ (kPa)	Cohesion, c (kPa)	Angle of friction, ϕ (°)	Angle of friction, ϕ (°)
12	94.0	176.0	256.0	13.3	39.0	41.0
15	97.0	178.0	251.0	21.3	37.5	39.0
24	81.0	158.0	243.0	0.0	39.0	37.0
25	80.0	155.0	235.0	1.7	38.0	36.0
28	85.0	160.0	242.0	5.3	38.0	38.0
32	90.0	170.0	249.0	10.7	38.5	39.0

Figure 6. Shear test results for the till samples from each of the six rock provinces, A/B–G.

Figure 7. Average shear stress vs. horizontal displacement for all samples categorised in each of the six rock provinces A/B–G for (A) 100 kPa, (B) 200 kPa and (C) 300 kPa normal stress.

Discussion

During the fieldwork there were some challenges regarding field interpretation for selecting the ideal sample sites. The tills were, for practical reasons, therefore chosen independently of genesis. However, genesis is known to influence the geotechnical properties of tills through its control on important characteristics such as stress history and particle size distribution. Also, post-depositional processes, such as wetting and drying, freezing and thawing, and downward percolation of fines, may further change the characteristics of the till (Boulton & Paul, 1976). As such variables are not accounted for in the sampling, they are sources of uncertainty for the representativeness of the samples regarding the area from which they are collected and, consequently, for the interpretation of the shear test results.

Furthermore, to distinguish a till deposit from an old, revegetated debris slide/flow deposit of till material was also rather challenging as the outcrops were commonly limited. According to Cammeraat & Rappol (1987), there is a possibility that till-like resedimentation products (debris flows) may be incorporated in the

sampling, as these can be difficult to differentiate from true tills in such cases where the exposure in the field is poor. Additionally, the degree of detailed mapping available from the Quaternary geology map (NGU, 2016c), especially in the transition zone from one deposit type to another, is quite poor in some areas. Although focusing on locations marked on the map as till, one cannot exclude the possibility that some of the samples are, in fact, from e.g., debris slide/flow deposits of till material. Yet, this latter potential source of uncertainty was regarded as having no significant influence, as the sampling itself disturbed the material.

The samples were not tested in their in situ state, as it is regarded as nearly impossible to collect perfectly undisturbed samples of till (e.g., Andresen, 1979). The disturbed samples were also prepared in the laboratory with sieving, splitting, drying and compacting before shear testing. According to Vanapalli et al. (1996), the laboratory preparation of samples must closely represent the physical conditions and the stress state conditions likely to occur in the field if a proper assessment of the shear strength parameters is to be achieved. For example, it is considered as practically impossible to obtain a soil

with zero saturation (Budhu, 2015), thus making the dried samples in this study not representable for normal Norwegian field conditions. As water is a main trigger for Norwegian debris slides and flows (e.g., Kronholm et al., 2007; Bargel et al., 2011), shear testing the samples in saturated conditions instead of in a dry condition would clearly be more appropriate for practical applications regarding slope stability assessment. In practice, however, including water in the testing would introduce a variable hard to control. This is due to significant differences of the geological parameters amongst the till samples, such as the particle size distribution, which could lead to, e.g., inhomogeneous saturation in the material and removal of fines. This, in turn, would potentially introduce a significant uncertainty to the results. Moreover, the relatively high levels of applied normal stress do not reflect current field conditions either. Consequently, these results are not likely to replicate the factual in situ/field conditions at the sample sites regarding till shear strength. Nevertheless, and as initially stated, the samples in this study were not tested with the purpose of replicating their field conditions, but with the same procedure for achieving a best possible, equal comparison basis. Hence, the aim of the study was to show if shear strength differences in relation to bedrock geology and the respective rock provinces do exist, regardless of individual field variables such as stress state, density and water content. Excluding these, although highly important, variables was considered necessary to be able to establish equal conditions of testing with a minimum of laboratory variables, and, accordingly, so that the results could be linked to the geological parameters (particle shape, size, etc.) of the samples.

Independently of uncertainties in the sampling and testing procedure, one should also evaluate the apparatus that was used. Regarding the use of a shear box apparatus, Das (2010) pointed out that the direct shear test method has some inherent shortcomings. For example, the reliability of shear box results may, in general, be questioned because the soil is forced to fail along the contact plane of the two halves of the box, rather than along a potentially weaker plane in the sample. However, the disturbed, heterogeneous material tested in this study should be relatively homogeneous when prepared in the sample box, thereby having no significantly weaker planes than the plane of split. Although the shear strength can be found by using other laboratory tests, such as the triaxial test, the direct shear test is considered to be the simplest and most economical for a dry sandy soil (Das, 2010), and, according to Lommler (2012), the commercial laboratory test of choice for granular soils.

Furthermore, according to e.g., Hooke (2005), detailed measurements often show that the variation of yield strength with effective normal pressure is not linear at low effective normal pressures, hereby giving rise to the terms 'true cohesion' and 'apparent cohesion'. While till may have true cohesion (Hooke, 2005; Hungr et al.,

2005), Cuffey & Paterson (2010) have pointed out that in shear at critical state the till has no true cohesion. Also, since these samples were prepared from disturbed material, there could certainly be no true cohesion (Hencher, 2012). Thus, the values of cohesion reported here are not true cohesion, but obtained values from the linear regression analysis.

As the reproducibility and repeatability of the results from shear tests may vary (e.g., Bareither et al., 2008a), six of the samples were retested to observe the possible change of results (Table 16). Bareither et al. (2008a) investigated the repeatability and reproducibility of direct shear tests on granular backfill material (sand). In that context, 'repeatability' meant testing identical items in the same laboratory by the same operator using both the same method and equipment (intra-laboratory), while 'reproducibility' meant testing the identical items with the same method, but in 10 other laboratories by different operators and equipment (inter-laboratory). In summary, the reproducibility varied significantly more than the repeatability. Regarding repeatability, the intra-laboratory tests on the angle of friction led to a SD of 0.1°. On the other hand, the inter-laboratory tests reported an average reproducibility of 8.8°, with a range of variability up to 18.8°. Compared to the present study, where the SD of the angle of friction ranged from 0.3° to 1.3°, the results were considerably better than the reproducibility results reported by Bareither et al. (2008a), although the repeatability was not as good. However, the reason for this diversity of results may be due to factors such as the difference of material used (relatively homogeneous sand vs. heterogeneous till material) as well as the number of tests performed (five vs. three). For these reasons, the results presented in Table 16 were considered satisfactory regarding the assessment of repeatability.

Moreover, on the issue of repeatability, studies by e.g., Feda (2002) and Hattamleh et al. (2010) have reported changes in shear strength due to particle crushing, which initially was also of some concern in this study, especially due to the necessary reuse of samples. Yet, when retesting six of the samples, the results were either equal to, higher and lower than the ones obtained from the main test. This strengthened the initial assumption that the samples would not be noticeably damaged or altered by reuse. Although there were some clear differences in some of the additional tests, it is difficult to isolate the exact reason(s), as these may be due to uncertainties in the shear box apparatus, geological parameters, procedure, operator, or a combination of these. Since the samples consist of gravel, sand and fines, emptying and refilling the sample box gives significantly more combinations of particle arrangements and interactions and, as a result, the repeatability should therefore vary more than retesting, e.g., a homogeneous sand.

Internationally, from shear tests on multiple tills from eastern England, Bell (2002) presented angles of friction

mainly ranging from approximately 20° to 30°, although the fraction of fines composed up to 60–80% of these tills. Iverson et al. (1994) reported an angle of friction of 31° in a till from Sweden, while a study by Rathbun et al. (2008) on two tills from Alaska and Ohio, USA, resulted in 31.4° and 29.4°, respectively. Others have reported that the angle of friction for till ranges between 35° and 45° (Koloski et al., 1989). Clearly, the angles of friction for tills may vary significantly. Considering the previously mentioned shear test studies of Norwegian tills, Lund (2013), Langåker (2014) and Langeland (2016) investigated one locality each, i.e., Nesbyen in Nes municipality, Buskerud county, Seim in Granvin municipality, Hordaland county, and Soknedal in Midtre Gauldal municipality, Sør-Trøndelag county, respectively. The angles of friction in the study by Langåker ranged between 38.9° and 42.7°, while the studies by Lund and Langeland resulted in 39.5° and 37.4°, respectively. Although the localities in these Norwegian studies do not overlap with any samples in this study, as well as not being tested with exactly the same procedure, the results are still quite comparable and substantiate the presented results. Especially the angle of friction from Langeland is near identical to the results from the three closest samples presented here, i.e., nos. 22, 23 and 33 (Table 10).

When incorporating bedrock geology into the results, both similarities and differences among the samples are shown within each rock province and when comparing the provinces to one another. It is well known that the compressive strength of different rock types varies (Afrouz, 1992; Nilsen & Palmstrøm, 2000; Waltham, 2009). According to the compressive strength description presented in NS-EN ISO 14689-1 (2004), the terms are 'extremely weak' (<1 MPa), 'very weak' (1–5 MPa), 'weak' (5–25 MPa), 'medium strong' (25–50 MPa), 'strong' (50–100 MPa), 'very strong' (100–250 MPa) and 'extremely strong' (>250 MPa). Accordingly, the tills are composed of rock types of varying compressive strengths. As initially mentioned, this difference in bedrock geology, including rock strength, should to some extent influence the till shear strength as well. In the following, an evaluation of the results is presented for each individual rock province, including a brief description of the typical rock strengths of the dominant rock type(s) within the respective provinces. Note that the compressive strength values given by Nilsen & Palmstrøm (2000) are average values from tests on Scandinavian rocks.

Province A/B: extrusive and plutonic rocks / sedimentary rocks in the Oslo region

When compared to the other provinces, the five samples collected from this province are geographically very close. In addition, the rock types of the respective sites are similar, being mainly dominated by syenite and quartz syenite, as well as granite and granodiorite. Palmstrøm (1997) also presented the compressive strength for several

Norwegian rock types, where syenite varies between 100 and 250 MPa. According to Nilsen & Palmstrøm (2000), the compressive strengths for granite and granodiorite are 169 MPa and 171 MPa, respectively, while monzonite has a compressive strength of 106 MPa. According to NS-EN ISO 14689-1 (2004), these rock types are termed as 'very strong'. On the other hand, phyllite and mica schist are listed with compressive strengths of 61 MPa and 71 MPa, respectively (Nilsen & Palmstrøm, 2000), and are termed as 'strong' rocks. However, the compressive strength of both phyllite and mica schist may also be as low as 20 MPa (Palmstrøm, 1997), thereby termed as 'weak' rocks. By solely comparing the compressive strength of these rock types with the till shear test results, in addition to the relatively small geographic area, one should expect intermediate to high shear strength and quite similar results, especially for sample nos. 5, 30 and 31. By comparison, sample no. 6, which contains a significant component of potentially weak mica schist and phyllite, should stand out as the weakest regarding shear strength. The results show a rather intermediate level of average max. shear stresses and angle of friction when compared with the other provinces (Table 15). Actually, the average angle of friction for this province is almost the same as province E. In addition, sample nos. 5, 31 and 32 show the most similar results, whereas no. 30 unexpectedly is the weakest due to the low angle of friction and registered shear stresses, even though the rock types are considered stronger than those in no. 6. This suggests that parameters other than rock (particle) strength may have a larger influence on the results in this particular case.

Province C: Precambrian basement

The six samples in this province are geographically the most scattered, extending from the southeastern part of Norway close to the Swedish border towards the southwestern part of the country. Various gneisses and granites dominate in five of the samples. According to Nilsen & Palmstrøm (2000), the compressive strength is 130 MPa for gneiss and 169 MPa for granite. It is important to note, however, that the designation 'gneiss' is used for rocks with potentially large mineralogical differences (Prestvik, 1995). Thus, the compressive strength for gneiss may vary from 80 MPa to 200 MPa (Palmstrøm, 1997). Due to the compressive strengths of these rock types, termed as 'very strong', one should expect a rather high till shear strength. As expected, on average this province scores quite high regarding max. shear stresses and the angle of friction. The average angle of friction is the third highest of all provinces (Table 15). However, the average is affected by the rather low angles of friction from sample nos. 28 and 29. The results show that sample nos. 4, 7, 8 and 9 are quite similar in having a high angle of friction, clearly differing from nos. 28 and 29, which, in turn, are quite similar to one another by having a lower angle of friction. These two southeastern located

samples are geographically close and relatively similar regarding rock types, but they also have some differences when compared to the remaining four samples. The differences of geography and rock types may partly explain the differences of results. For example, one possible explanation could be the fact that this southeastern region is known for having several mylonite zones, where the rocks have been exposed to both plastic deformation and crushing (Ofte Dahl, 1981). One cannot exclude that such processes may have influenced these two samples regarding geological parameters such as rock strength and particle size distribution. However, only sample no. 29 seems to lie within such a mylonite zone, and it is also slightly stronger than sample no. 28, thereby reducing the likelihood that the mylonite zones have affected the material in a way that influenced till shear strength.

Province D: Caledonian rocks, overthrust sheets of Precambrian rocks

As for province A/B, the samples from this province are also geographically fairly close. Apart from sample no. 13, which is solely represented by anorthosite, the remaining samples are dominated by mangerite and gabbro, as well as gneiss and amphibolite. Nilsen & Palmstrøm (2000) list the compressive strength as 248 MPa for gabbro and 107 MPa for amphibolite. However, gabbro may have a compressive strength up to 300 MPa (Palmstrøm, 1997), thus classified as 'extremely strong'. Moreover, anorthosite has a compressive strength of 157 MPa (Nilsen & Palmstrøm, 2000). By only focusing on rock strength, the general till shear strength should be high in this case. As expected, the average max. shear stresses and angle of friction are high. The average angle of friction is higher than for province C and ranks second of all provinces (Table 15). However, this province has two rather unexpected extremes. While sample nos. 10, 13 and 14 are relatively similar, no. 11 stands out as clearly weaker and no. 12 as clearly stronger. In addition, sample nos. 11 and 12 are geographically close, and they are also quite alike regarding rock types. This difference can hardly be explained by rock strength and, as suggested for province A/B, such unexpected results imply that geological parameters other than rock strength may have a larger influence on till shear strength.

Province E: Caledonian rocks, metamorphic and igneous rocks

The diversities regarding rock types in this province are larger and more complex than any other province, e.g., mica schist, greenstone, phyllite, amphibolite, quartzite, slate and limestone. Therefore, the span of compressive rock strengths differs accordingly, and one would expect a rather greater variety in the shear test results. Quartzite is significantly stronger than phyllite and mica schist, i.e., 172 MPa, while greenstone is quite similar to amphibolite

with a compressive strength of 105 MPa (Nilsen & Palmstrøm, 2000). Slate is listed by Waltham (2009) to have an average compressive strength of 90 MPa. Limestone has a compressive strength of 74 MPa, while mica gneiss is similar to that of slate, i.e., 89 MPa (Nilsen & Palmstrøm, 2000). Hence, this large variation of rock types and corresponding compressive strengths would imply a variation of the shear test results, but preferably in the intermediate to low levels due to the rather large components of potentially weak mica schist and phyllite. Although the results show intermediate to low levels of average max. shear stresses and angle of friction when compared with the other provinces (Table 15), many of the sample results are actually quite similar, thereby making it difficult to link the individual sample results directly and solely to the associated rock strengths.

Province F: Caledonian rocks, overthrust sheets of sandstone and schist

The samples in this province are relatively homogeneous regarding rock types. Sandstone is by far the most dominant, with only smaller amounts of other rock types such as conglomerate, quartzite and mica schist represented. Sandstone has a compressive strength of 147 MPa (Nilsen & Palmstrøm, 2000), although it may vary between 75 MPa and 160 MPa (Palmstrøm, 1997), or even be as low as 30 MPa according to Waltham (2009). Since Norwegian sandstone, which is presumably 'strong' to 'very strong', is dominant in all five samples, this province should show intermediate to high shear strengths and low diversities in the range of results when compared with the other provinces. Although the individual angles of friction and shear stresses are quite similar, the results unexpectedly also show that this province on average has the lowest angle of friction and the lowest max. shear stresses (Table 15). If the till shear strength is to be solely related to the respective rock strengths, the results indicate rock strengths at the lower end of the scale listed by Palmstrøm (1997) or Waltham (2009).

Province G: Caledonian rocks, Precambrian basement, locally affected by the Caledonian orogeny

This province is also quite homogeneous regarding rock types, where various gneisses are dominant. Even though gneisses may vary significantly in composition (Prestvik, 1995), they are generally regarded as 'strong' to 'very strong' rocks (Nilsen & Palmstrøm, 2000; Waltham, 2009). As expected, the angles of friction are both high and similar to one another, and, in fact, they are on average the highest measured of all provinces (Table 15). On average, this province also has the highest registered max. shear stresses. However, the two northwesternmost samples (nos. 18 and 19) are noticeably weaker than the three remaining samples, but at the same time quite

similar to each other. An explanation for this difference is uncertain, but may partly be due to geographical and mineralogical variations of the gneisses.

Summary of discussion

When analysing and comparing individual samples to one another within their respective provinces, it is rather difficult to establish a clear and direct relationship between the sample strength and their rock type(s), and, especially, to the associated typical rock strength. However, an important uncertainty on this matter is that the listed compressive strength values from the literature are average values, including a significant range, which are not necessarily representative for the samples in this study. On an individual sample level there are some results that do not relate, as one would expect, to bedrock geology and rock strength. Some samples that are dominated by the same rock type(s) and that are geographically close, show both quite different and similar results, e.g., sample nos. 11 and 12, and nos. 16 and 17, respectively. Consequently, and according to the literature, rock (particle) strength cannot be the only parameter influencing till shear strength. As initially mentioned, this implies that other geological parameters are involved, such as particle size distribution and particle shape, which are also known to have an effect on soil shear strength (e.g., Cornforth, 1973; Yagiz, 2001). If the study rather focused on shear testing the samples in a densely packed state, which most likely would introduce dilatancy and the potential of particle crushing, rock strength would possibly be more directly influential. However, on a more generalised level when comparing each of the provinces to one another, there are noticeable differences and a relation to bedrock geology, e.g., in Fig. 7A–C. For the first half of horizontal displacement, the results show that for all three tests the provinces C, D and G are clearly different from A/B, E and F, which are consistently lower regarding shear stresses in the same displacement interval. This difference may be due to the individual province and sample differences regarding the geological parameters, the fraction size limit (<16 mm), or a combination of these, but it basically seems that provinces C, D and G initially exert more shear resistance than the other three provinces. Such noticeable differences, in turn, may influence the potential of triggering debris slides and flows. The differences, however, are eventually reduced, and the levels of shear stress become more similar towards the end of the displacement. Although the general trend of the shear stress levels of the provinces seems to remain towards the end, the reduction implies that the effect of the different geological parameters in the respective provinces eventually becomes less influential.

Simultaneously, it is necessary to point out the main possible sources of uncertainty regarding the samples. A key aspect to address is that the number of samples may

be considered too few to draw a definite conclusion on the levels and differences of shear strength of Norwegian tills, especially with regard to the respective rock provinces. Furthermore, the analysed till samples were independent of their genesis, i.e., no distinction between subglacial till and supraglacial till was made. Another important aspect on this matter is that the changing directions of ice motion and/or climatic fluctuations may have resulted in many tills that are a mixture of glacially eroded rock material and redeposited older sediments (Gillberg, 1977). This is not accounted for, and even if the till material was made solely from erosion of unweathered bedrock material, there is also an unknown variable regarding glacial comminution ('terminal grades') of mineral grains, e.g., between grains from unmetamorphic clastic sedimentary rocks and grains from crystalline rocks. For this example, the latter are to a large extent unstable when subjected to glacial and other mechanical crushing processes (Haldorsen, 1978). Also, due to the relatively passive transport of supraglacial tills, the associated occurrence of comminution is known to be minor (Boulton, 1978). Accordingly, variables such as transport are likely to have influenced both the particle shape and particle size (Clark, 1987), as well as the actual percentage share of different rock-type fragments in the tills. This may especially be the case for samples such as no. 4 in province C, consisting of both sandstone and gneiss. It must be emphasised, however, that the given percentage shares of rock types comprising the till samples are only assumed on a theoretical basis. Consequently, the percentage share of the area of rock type(s) surrounding each sample site is based on an approximate measurement and calculation of the bedrock geology map, which is a planar projection not accounting for topography. Although Ehlers (1983) pointed out that different studies (e.g., Perttunen, 1977) have shown that the Scandinavian tills closely reflect the composition of the local bedrock, the bedrock geology can change over short distances, thus introducing significant uncertainty to this theoretical approach. In addition, the ice-flow directions and the chosen maximum transport distance of five kilometres are also simplifications likely to introduce some uncertainty into the results.

Conclusions

The main shear test performed on 33 near-surface, genetically independent, till samples shows a variation in the registered maximum shear stress up to approximately 16%, whereas the angle of friction varies by 5.5° (36.0°–41.5°) with an average of 38.4°. In general, the results differ more in the main test than in the additional tests for repeatability assessment. When observing the comparison of average angles of friction for the different provinces (Table 15), as well as average shear stresses (Fig. 7A–C), a

clear relation to bedrock geology and rock provinces does appear. This is particularly noticeable in the first half of the horizontal displacement length, where the differences of shear stress are the greatest. The average results show that for all three main tests the provinces C, D and G display higher shear stress and angle of friction than A/B, E and F, which are consistently lower. Generally, provinces of 'strong rocks' have a higher registered shear stress level and angle of friction than provinces of 'weak rocks'. However, these relationships are not so clear on an individual sample level when focusing solely on rock strength, suggesting that rock (particle) strength is not directly the governing parameter of till shear strength. Rock strength would possibly be more influential on the results if the study rather focused on shear testing the till samples in a densely packed state, which most likely would introduce dilatancy and potential particle crushing. For this case, it is considered more likely that rock strength, being a result of variations of rock types throughout the rock provinces, rather indirectly influences till shear strength due to its effect on, e.g., particle shape and size distribution, and, furthermore, that these geological parameters are more directly influential when shear testing the samples in such a loosely packed state. For the last half of horizontal displacement, however, all six provinces show a relatively similar level of average shear stress. Even though the general trend regarding the provinces of high shear stress and low shear stress seems to continue towards the end, it appears that the geological parameters influencing the till shear stress of the respective rock type(s) eventually become less influential.

In summary, due to the noticeable differences of shear stress and angle of friction, especially between the 'strongest province', G, and the 'weakest province', F, this study demonstrates a relation between till shear strength and bedrock geology. Although not conclusive, this suggests that some rock provinces may, solely on the basis of bedrock geology and associated geological parameters, be more prone to debris slides and flows than others. However, it must be emphasised that what makes an area prone to debris slides and flows also depends on many other parameters, i.e., the terrain, hydrological conditions, vegetation, etc. (Chatwin et al., 1994; Bargel et al., 2011). In addition, the number of samples tested is relatively few and they are geographically widely scattered. The samples are also independent of genesis, which is known to influence several of the characteristics of tills, and this is not accounted for. Moreover, the testing procedure and results do not replicate individual field/in situ conditions at the sample sites, as the samples were disturbed when collected and prepared for the purpose of achieving a comparison basis with a minimum of laboratory variables. Hence, the presented results should be regarded as preliminary. For this case, additional testing should be performed if one is to improve the statistical base of the shear strength of Norwegian tills and provide a more thorough evaluation of its relation to bedrock geology.

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