REPORT

SP8-GEODIP

CORRELATIONS BETWEEN SHEAR WAVE VELOCITY AND GEOTECHNICAL PARAMETERS IN NORWEGIAN CLAYS

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Summary

The purpose of this study is to present guidelines and correlations to assists geotechnical engineers in estimating V_s profiles in Norwegian clays in the absence of site-specific data. For this, a database of *in situ* V_s measurements and standard geotechnical engineering material properties for Norwegian clays has been established. The database allowed the development of several empirical correlations between in situ V_s and basic soil properties, cone penetration parameters, undrained shear strength and 1D compression parameters. Based on the results from regression analyses, we recommend the use of empirical functions based on cone penetrometer data to determine the best estimate *in situ* V_s of Norwegian clay when *in situ* measurements of V_s at the site are not available. Relationships based on undrained shear strength from CAUC or DSS tests can also be used in practice.

In general, it is recommended that engineers consider all available data including available relationships, *in situ* measured V_s profiles, and site-specific geotechnical data. The use of correlations in geotechnical engineering should be limited to the conditions for which they were developed and calibrated. The recommendations presented in this report should be used in conjunction with the engineer's own experience and engineering judgment. Site-specific correlations may be developed based on a limited number of site-specific V_s measurements and using a similar functional form as those presented in this report.

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1 Introduction

1.1 Background

Characterization of the stress-strain behaviour of soils is an integral part of many geotechnical design applications, including site characterization, settlement analyses, seismic hazard analyses, site response analysis and soil-structure interaction. The shear modulus (G) of geomaterials is highly dependent upon strain level. The small-strain shear modulus (G_{max} or G_0) is typically associated with strains on the order of 10^{-3}% or less. With information of G_{max}, the shear response at various level of stain can be estimated using published modulus reduction curves (i.e. G/G_{max}).

According to elastic theory, G_{max} may be calculated from the shear wave velocity using the following equation:

[1] $$G_{max} = \rho V_s^2$$

where G_{max} is the shear modulus (in Pa), V_s is the shear wave velocity (in m/s), and ρ is the density (in kg/m^3).

G_{max} and V_s are primarily functions of soil density, void ratio, and effective stress, with secondary influences including soil type, age, depositional environment, cementation and stress history, see for example (Hardin and Drnevich, 1972). The effect of several parameters on V_s is summarized in Table 1.

G_{max} can be measured in the laboratory using a resonant column device or bender elements. As suggested by (Kramer, 1996), while the void ratio and stress conditions can be recreated in a reconstituted specimen, other factors such as soil fabric and cementation cannot. Laboratory testing requires very high-quality, undisturbed samples which is often a challenging and expensive task in soft and sensitive clays. Additionally, laboratory tests only measure G_{max} at discrete sample locations, which may not be representative of the entire soil profile.

Unlike laboratory testing, in situ geophysical tests do not require undisturbed sampling, maintain *in situ* stresses during testing, and measure the response of a large volume of soil. *In situ* measurement of V_s has become the preferred method for estimating the small strain shear properties and has been incorporated into site classifications systems and ground motion prediction equations worldwide.

Table 1: Effects of various parameters on Gmax and Vs after (Dobry and Vucetic, 1987).

Factor/Parameter	Influence on G_{max} and V_s
Overburden stress (p'_0)	Increases with p'_0
Void ratio (e)	Decreases with increased void ratio
Age of deposit	Increases with age
Cementation	Increases with cementation effects
Overconsolidation ratio (OCR)	Increases with OCR
Strain rate ($\dot{\gamma}$) or frequency of cyclic loading	Increases with $\dot{\gamma}$

1.2 Eurocode 8 – Soil conditions and seismic design criteria

The seismic design criteria in Eurocode 8 [Norsk standard, NS-EN 1998-1:2004+NA:2008] classifies sites based on V_s of the top 30 m of the soil profile (V_{s30}). Sites are divided into the seven categories (soil profiles types A through E in addition to S_1 and S_2) presented in Table 2.

For site classification, V_{S30} is calculated as the time for a shear wave to travel from a depth of 30 m to the ground surface, not the arithmetic average of V_S to a depth of 30 m. As shown in Equation (2), the time-averaged V_{s30} is calculated as 30 m divided by the sum of the travel times for shear waves to travel through each layer. The travel time for each layer is calculated as the layer thickness (d) divided by V_s.

[2] $$V_{s30} = 30/\sum(\frac{d}{Vs})$$

For example, the V_{s30} for a soil profile containing 18 m of soft clay (V_s = 80 m/sec) over 12 m of stiff clay (V_s = 250 m/sec) would be calculated:
30 / (18 / 80 + 12 / 250) = 110 m/sec. The time-average method typically results in a lower V_{s30} than the weighted average of velocities of the individual layers: (80 · 18 + 250 · 12) / 30 = 148 m/sec.

Table 2: Ground types taken from Eurocode 8 [Norsk standard, NS-EN 1998-1:2004+NA:2008]

Ground type	Description of stratigraphic profile	Parameters		
		V_{s30} (m/s)	N_{SPT} (blows/30cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface	> 800	–	–
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of m in thickness, characterized by a gradual increase of mechanical properties with depth	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of m	180 – 360	15 – 50	70 – 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with V_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with V_s > 800 m/s			
S_1	Deposits consisting – or containing a layer at least 10 m thick – of soft clays/silts with high plasticity index (PI>40%) and high water content	< 100 (indicative)	–	10 – 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1			

For cases where measured V_s data is not available, alternative site class definitions are provided in terms of standard penetration test (SPT) resistance for cohesionless soils and undrained shear strength for cohesive soils. However, the SPT is not a common tool used in practice in Norway and/or other Nordic countries. Additional criteria, such as plasticity index, water content, organic content, collapse potential, and liquefaction potential, must also be considered when assigning a soil profile type. In addition to site classification, V_s may be required for site-specific seismic evaluation or dynamic analysis when required by the seismic design criteria.

1.3 Objectives and scope of work

The combination of geotechnical and geophysical methods for characterization of soft and sensitive clay deposits has advanced considerably over the last decade thanks to improve technology, field equipment and software developments. Site-specific measurements of V_s is the preferred method of determination of V_s and should be used whenever practical and/or whenever project economy allows it. A short overview of different geophysical methods for assessing V_s is presented in chapter 2.

In the absence of site-specific measurement, guidelines for estimating the V_s profiles based on correlations with *in situ* penetration tests, soil index parameters and undrained

shear strength may be used, recognizing that these indirect methods introduce greater uncertainties. The main objective of this report is to present such guidelines for estimation of V_s in Norwegian clays. This work is based on a database of 29 Norwegian clay sites where V_s and soil geotechnical properties were gathered for correlation purposes. The developed Norwegian clay database is presented in Chapter 3, while relationships between V_s and index properties, cone penetration parameters, undrained shear strength and 1D compression parameters are presented in Chapters 4-7. Note that the relationships presented herein can be used to evaluate either V_s from a given soil property, or the way around to evaluate soil properties from V_s.

It is recommended to use multiple indirect methods when possible in selection of the design V_{s30} when direct measurements of the V_s profile are not available. Engineering judgment should also be used to assess (1) the quality of data, (2) agreement between methods and (3) the potential impacts of under-predicting or over-predicting V_{s30} on structural performance.

2 Techniques used for the measurement of shear wave velocity

Shear modulus is directly linked to a material's stiffness and is one of the most critical engineering parameters. Seismically, shear-wave velocity (V_s) is its best indicator. The major advantage of field measurements of V_s is that the soil is tested in its natural state, thus mitigating the dramatic effects of sample disturbance caused by drilling, tube insertion, extraction, transportation, storage, trimming, and reconsolidation. Figure 1 follows the stress history of a soil sample from sampling to reconsolidation for testing. The final state can sometimes be significantly different than the real soil *in situ*. With field geophysics, larger volumes of soil can be tested, in many cases more rapidly and at lower cost than comparable lab tests.

Geophysical methods can be divided into two categories: invasive and non-invasive. Invasive methods require drilling into the ground. Common invasive methods include: downhole logging, crosshole logging, suspension logging, seismic dilatometer (SDMT) and the seismic cone penetration test (SCPT). The seismic dilatometer (SDMT) is the combination of the standard flat dilatometer (DMT) with a seismic module for measuring the shear wave velocity V_S (Marchetti et al. 2008). The SDMT method has become very popular in some countries (e.g. U.K., Italy) because of well-developed software. The SCPT is a modified downhole measurement in conjunction with the conventional cone penetration test (CPT). The SCPT has become more common in recent years because it is a relatively rapid and cost-effective method of measuring V_s. Non-invasive geophysical methods include: spectral analysis of surface waves (SASW), multichannel analysis of surface waves (MASW), seismic refraction, and seismic reflection.

In the following sections only the different techniques used for the measurement of shear wave velocity included in the Norwegian clay database are presented.

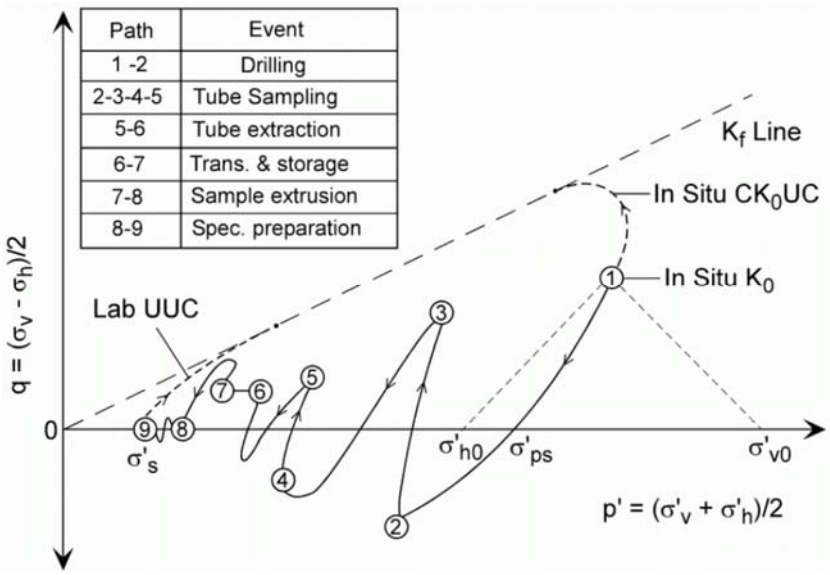

Figure 1: Hypothetical stress history caused by tube sampling of a low OCR clay, from (Ladd and De Groot, 2003).

2.1 Invasive methods

2.1.1 Seismic cone penetrometer (SCPTU)

The SCPTU was first introduced in 1984 at the University of British Columbia, see (Rice, 1984), (Campanella et al., 1986) and (Robertson et al., 1986). According to (Lunne et al., 1997b) the technique combines versatility and simplicity together with the speed and efficiency similar to the traditional standardized cone penetration test. The components of the typical test equipment and the basic procedures for the SCPTU have changed little since its development. A cone penetrometer, including one or more horizontally aligned seismic sensors, is pushed into the ground vertically at a rate of 20 mm/s. The seismic measurements can be made using a single receiver and the pseudo-interval method, or with two receivers and the true-interval method (Figure 2). During penetration, readings of tip stress, friction sleeve stress, and pore water pressure are also taken at given intervals. The pseudo interval or true-interval seismic signals are only recorded during pauses in penetration, commonly every 0.5 or 1.0 m. In usual practice, a horizontal beam or plate coupled to the ground surface by the weight of a support vehicle or the testing vehicle is the source of the seismic energy. The beam is struck on end with a hammer to generate horizontally polarized vertically propagating shear waves that can be detected by the horizontal receiver(s) within the cone penetrometer embedded below. The velocity is determined from the travel-time differences between recorded waves and the difference in the assumed travel path length for different receiver depths.

This is a cost effective method for characterizing subsurface profiles, capable of measuring five separate parameters including, tip resistance, local friction, pore pressure upon penetration, time for porewater dissipations, and V_s, all within the same test.

A sketch illustrating how to determine interval shear wave velocity from SCPTU is provided in Figure 2. Inverting travel time differences into interval shear wave velocities inherently assumes straight travel paths, whereas typically, ray-bending will occur due to vertical contrasts in acoustic impedance and according to Snell's law. However, with short horizontal offset between the source and the drill string, this effect can be largely ignored. Furthermore, in particular the shallower recordings would be affected whereas those at depth would be virtually free of ray-bending.

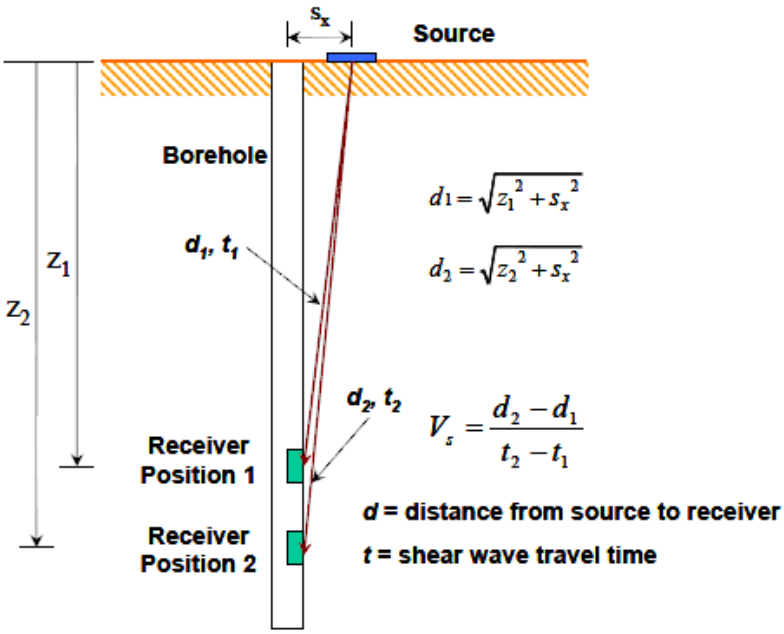

Figure 2: Sketch of the principle used to determine shear wave velocity for the interval between two geophones and a seismic source at the surface, see (Liao and Mayne, 2007). Note that, for the equipment used in this study, only one geophone is present and time histories are compared from signals generated before and after pushing the cone penetration rod in the soils over a given distance.

Three different techniques can be used to determine *in situ* average shear wave interval velocity between two adjacent geophone locations, being (i) the cross-over method; (ii) the cross-correlation method; and (iii) the phase-shift method. For more information about these methods the reader is referred to (NGI, 2011). In addition, the timing of the shear wave arrival at a given receiver also provides information on the average shear wave velocity between the source and the receiver. However, interval velocities are more informative than average velocities with depth.

In principle, it is advantageous and recommended to have multiple geophones installed in the cone penetration rods, and use identical shots to determine shear wave velocities

for the intervals in between the receivers. This would make the shear wave velocities less dependent on the source signature. In case the source signature is poor (low repeatability), comparing time histories from different blows are less reliable. Having multiple geophones also alleviates potential issues with inaccuracies of the target depths. In addition, sampling interval in time domain must be sufficiently high to allow higher-resolution accuracy in finding the time lags or optimal phase shifts in the analysis.

The SCPTU method was used for collecting shear wave velocity information at 7 of the sites presented in the database.

2.1.2 Cross-hole test (CHT)

The cross-hole test is often considered the reference standard by which other in-situ shear wave velocity tests are compared. The tests are performed in a series of 2 or more cased boreholes. A borehole seismic source generates waves that propagate past receivers at the same depth in adjacent boreholes (Figure 3).

The velocity is determined from the travel time of the waves over the distances between boreholes. The layering is considered to be essentially horizontal between the boreholes and the measured velocity is applicable to a particular layer. The classical reviews of cross-hole test procedures can be found in (Hoar and Stokoe, 1978) and (Woods, 1978). One major advantage of cross-hole testing is the direct measurement through only the desired material of a particular select layer. Because of the direct measurement and the reasonable certainty of the travel path of the source waves, the results are considered to be accurate. The test can be conducted in soil and or rock materials, and testing depths can be taken quite deep, up to 300 m or more. The greatest disadvantage of CHT is the need for multiple cased and grouted boreholes with accurate inclination records. The results are sensitive to variations in the spacing between the boreholes. As a consequence, the CHT is slow, time consuming, and very expensive. Internationally, the large costs associated with such test has discouraged V_s profiling by CHT for routine small to medium projects. In the database presented in chapter 3 and 4, the CHT was used at 5 of the sites (i.e. Berg, Esp, Tiller, Danviksgata and Bothkennar).

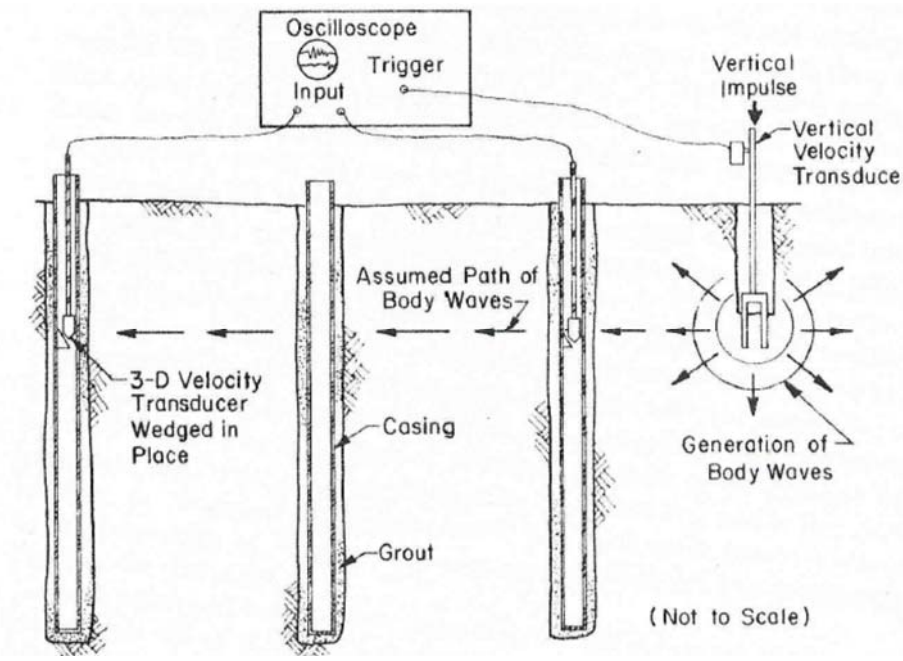

Figure 3: Crosshole test configuration, from (Hoar and Stokoe, 1978).

2.2 Non-invasive methods

2.2.1 SASW technique

In the early 1980s the widely used spectral analysis of surface waves (SASW) method was developed by (Heisey et al., 1982) and (Nazarian and Stokoe, 1984). The SASW method uses a single pair of receivers that are placed collinear with an impulsive source (e.g., a sledgehammer). The test is repeated a number of times for different geometrical configurations. (Crice, 2005) acknowledged the usefulness of SASW but suggested that solutions are neither unique nor trivial and that an expert user is required for interpretation. (Lo Presti et al., 2003) and (Soccodato, 2003) compared V_s derived from SASW with that obtained from other techniques for Pisa clay and Fucino clayey soil, respectively. Reasonable agreement was found in both cases. The SASW method was used for recording and processing of surface wave data for 4 sites discussed in this report.

2.2.2 MASW technique

The multichannel analysis of surface waves (MASW) technique was introduced in the late 1990s by the Kansas Geological Survey, see (Park et al., 1999), to address the problems associated with SASW. This method utilizes the dispersion property of surface waves for the purpose of V_S profiling in 1D (depth) or 2D (depth and surface location) format. Basically it is an engineering seismic method dealing with frequencies in a few to a few tens of Hz (e.g., 3–30 Hz) recorded by using a multichannel (24 or

more channels) recording system and a receiver array deployed over a few to a few hundred meters of distance (e.g., 2–200 m); i.e. similar to those used in conventional seismic reflection surveys. The investigation depth is usually shallower than 30 m.

The MASW method has improved production in the field owing to multiple transducers and improved characterization of dispersion relationships by sampling the spatial wave field with multiple receivers. Advantages of this method include the need for only one-shot gather and its capability of identifying and isolating noise. Also, its ability to take into full account the complicated nature of seismic waves that always contain noise waves such as unwanted higher modes of surface waves, body waves, scattered waves, traffic waves, as well as fundamental-mode surface waves. These waves may often adversely influence each other during the analysis of their dispersion properties if they are not properly accounted for.

(Crice, 2005) illustrates how MASW survey data can be reliably interpreted by computer software without human intervention. The authors have found that this is only accurate for simple soil profiles. Significant user experience and intervention are required for more complex profiles, as the inversion formulation in MASW can suffer the same uniqueness problems as in SASW. In the view of the authors an informed user is certainly important for MASW data analysis. The MASW method was used for recording and processing of surface wave data for nearly all sites presented in the database (i.e. 28 out of 29). Results presented in Appendix B show that MASW can reliably measure Vs down to 15-20 m in clay deposits.

An impulsive source (sledgehammer) was used to generate the surface waves at the Norwegian clay sites. Seismic data were recorded using an RAS-24 seismograph and the corresponding Seistronix software. Typically the test configuration comprised either twenty-four 10 Hz geophones or twelve 4.5 Hz geophones spaced at 1 m centres over the survey length. Although the 4.5 Hz geophones were used on the sites with the softest soils, it was found that they provided little advantage over the higher frequency instruments. For the 10 Hz geophones, the lower frequency level was not limited by their natural frequency, and they could detect signals as low as 5 Hz. With the 4.5 Hz geophones, the lowest recordable frequency was 2–3 Hz. A similar finding is reported by (Park et al., 2002), who discuss optimum acquisition parameters for MASW surveying.

With the multichannel approach, dispersion properties of all types of waves (both body and surface waves) are imaged through a wavefield-transformation method that directly converts the multichannel record into an image where a specific dispersion pattern is recognized in the transformed energy distribution. Then, the necessary dispersion property (like that of the fundamental mode) is extracted from the identified pattern. All other reflected/scattered waves are usually automatically removed during the transformation. The entire procedure for MASW usually consists of three steps: (1) acquiring multichannel field records (or shot gathers); (2) extracting dispersion curves (one from each record); and (3) inverting these dispersion curves to obtain 1D (depth) V_S

profiles (one profile from one curve). Different codes and software can be used for the fundamental mode inversion to obtain V_s profiles. The tools used in this study are Surfseis, WinMASW and an NGI in-house inversion code.

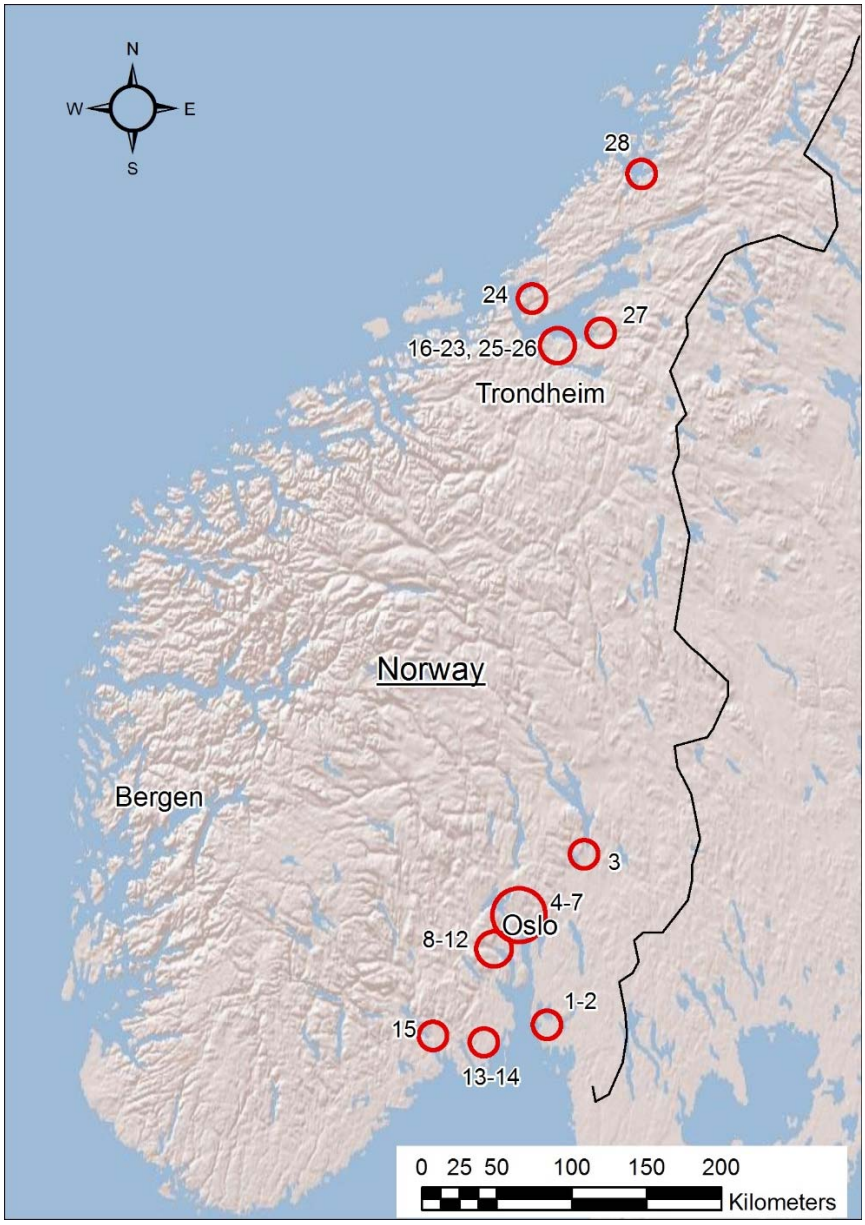

Figure 4: Overview map showing location of study sites included in database.

3 Test sites and soil properties included in the database

In situ shear wave velocity measurement has been carried out at a few Norwegian clay sites during the last decades for research purposes and/or as a part of construction projects. Source of existing data includes e.g. papers by (Long and Donohue, 2007) and (Long and Donohue, 2010), (L'Heureux et al., 2013) and the PhD thesis of (Langø, 1991). In this report we assembled previously published information with new field data in the Norwegian clay database. The data originates from a total of 29 sites as summarized in Table 3. Out of these sites, 15 are located in south-eastern Norway while 13 are in mid Norway (Figure 4). The last site included in the database is the Bothkennar clay site in Scotland where much work has been carried out over the last 30 years (including testing of block samples by NGI), see for example (Long et al., 2008). The reader is referred to Appendix A and B for detailed map locations and thorough description of each site, respectively.

The compiled database contains index and engineering properties obtained from classification tests, strength tests and consolidation tests. The database includes index properties such as total unit weight, water content, clay content, remoulded shear strength, sensitivity and Atterberg limits. Also, engineering properties such as undrained shear strength derived from CAUC, CAUE, DSS and in situ vane tests, net cone resistance, in situ effective vertical stress and 1D compression parameters based on the classical Janbu theory, see (Janbu, 1963) and (Janbu, 1969). Comparative in situ V_s data was acquired at each site using either SASW, MASW, SCPTU, or a combination of these (see Table 3). A summary of index properties and in situ shear wave velocity at the different study sites is given in Table 4.

The Norwegian clays in the database are of marine or glacimarine origin. Natural water content (w) data range between 20 and 80% with most of the data in the range between 40 to 50% (Figure 5a). The plastic index (I_p) being define as the difference between the liquid and plastic limits is presented in Figure 5b. Most of the plasticity index data vary between 5 and 20%. The clay content of the soil tested ranges from 10 to 70% with most of the data in the range between 30 to 50% (Figure 5c).

Due to the isostatic uplift and resulting emergence of the marine and glacimarine deposits during the last c. 10,000 years, fluxes of fresh groundwater through the clay deposits have led to leaching of the salts within the grain structure of the material. According to Rosenqvist (1953), such process is the main factor affecting the sensitivity of the clays. Sensitivity is defined as the ratio of the undrained peak shear strength over the remolded shear strength. In the database, the sensitivity of the clays ranges between 0 and 240 with most of the data in the interval 0-20 (Figure 5d).

The histogram of sample depth for the various clay samples in the database is presented in Figure 6a and the corresponding vertical *in situ* effective vertical stress for these depths is shown in Figure 6b. The effective vertical stress in the database varies between 10 and 240 kPa with the highest number of observations at around 100 kPa

corresponding to a depth of approximately 6-7 m below ground surface. Most of the clays have developed some apparent overconsolidation due to aging. The overconsolidation ratio (OCR) data range between 1.0 and 8 with most of the OCR data falling between 1.5 and 2.0, indicating that most of the soil samples in the database are normally consolidated to lightly over consolidated (Figure 7a). Hence, the developed correlations in chapter 7 may not be valid for heavily overconsolidated clays. The undrained shear strength data from CAUC triaxial tests concentrates in the range 25-60 kPa whereas results from CAUE and DSS tests are mostly below 50 kPa (Figure 7b).

Table 3: Summary of site surveyed

no.	Location	Site	Soil type	Technique	References for sites
1	*Østfold*	Onsøy	soft clay	SCPT / MASW	Eidsmoen et al. (1985), Lunne et al. (2003)
2		Seut bridge	soft organic clay (quick)	MASW	APEX files, Multiconsult files
3	*Akershus*	Eidsvoll	firm to stiff clay (silty)	MASW	Karlsrud et al. (1996), (2005), Karlsrud and Hernandez-Martinez (2013), Lunne et al. (1997a), (2006)
4		Hvalsdalen	firm to stiff clay	MASW	As Eidsvoll
5		Skøyen - Asker	Very soft clay (quick)	MASW	NGI files, e.g. NGI 990032-1
6		RVII	soft clay	MASW	Long et al. (2009), Hagberg et al. (2007)
7	*Oslo*	NGI car park	soft clay	MASW / SASW	NGI files, Kaynia and Cleave (2006)
8	*Buskerud-Drammen*	Danviksgata / Museum-park	soft clay	SCPT / MASW / Raleigh / CHT	Lunne and Lacasse (1999), Eidsmoen et al. (1985), Butcher and Powell (1996), BRE (1990)
9		Lierstranda	soft clay	MASW / Raleigh	Lunne and Lacasse (1999), Lunne et. al (1997a)
10		Hvittingfoss	Soft to frim quick clay	SW inversion (MASW) / SCPTU / Seismic reflection	Sauvin et al. (2013a), Sauvin et al. (2013b), Sauvin et al. (2014)
11		Smørgrav	Soft (quick) clay	MASW	Donohue et al. (2009), Donohue et al. (2012), Pfaffhuber et al. (2010)
12		Vålen	Soft clay	MASW	Sauvin et al. (2011)
13	*Vestfold*	Farriseidet	Organic quick clay	MASW	NGI files
14		Månejordet	Silty quick clay	MASW	Statens vegvesen / UCD files
15	*Telemark*	Skienselven	Soft to firm quick clay	MASW	NGI files e.g. 20011544-1, Feb. 2003
16	*Trondheim*	Tiller	soft to firm (quick) clay	MASW / SASW / SCPTU / CHT	Gylland (2013), Sandven et al. (2004) Sandven (1990), NGI files, MSc. Henrik Takle Eide.
17		Berg	firm clay	MASW / CHT	Rømoen (2006), Westerlund (1978)
18		Esp	Soft to firm (quick?) clay	MASW / CHT/SCPTU	King (2013), Montafia (2013), Knutsen (2014), Hundal (2014), NGI files
19		Klett (South)	Soft silty (quick) clay	MASW / SCPTU	APEX, Multiconsult and NGI files

20		Dragvoll	Very soft quick clay	MASW, SW inversion	(Montafia, 2013), (Pasquet et al., 2014), Karl Fredrik Moe?, Tonje Eide Helle?,
21		Rosten	Soft clay	MASW	NGI files
22		Saupstad	Firm to quick clay	MASW	NGI files
23		Eberg	soft organic clay	SASW / Seismic ref.	Røsand (1986), Sandven (1990), Langø (1991)
24		Hoseith	Quick clay (silty)	MASW	APEX and Multiconsult files, Trondheim kommune
25		Okstad	Stiff, silty clay	MASW	APEX and Multiconsult files, Trondheim kommune
26	*Rissa*	Rein kirke	Soft and quick clay	MASW	Sauvin et al. (2013b), (Aasland, 2010), (Kornbrekke, 2012)
27	*Stjørdal*	Glava	Firm clay	MASW / SASW	Sandven (1990), Sandven and Sjursen (1998)
28	*Namsos*	Kattmarka	Layered soft clay	MASW	NGI/NTNU data
29	*Scotland*	Bothkennar	soft clay / silt	SCPT / SDMT / MASW / CSW CHT	See Géotechnique, No. 2 1992. For summary of V_s values see Long et al. (2008)

Table 4: Summary of soil properties at study sites.

	Site	w (%)	ρ (Mg/m^3)	Clay (%)	I_p (%)	S_t	OCR	V_s (m/s)
1	Onsøy	60 - 65	1.635	40 - 60	33 - 40	4.5 - 6	1.5 - 1.3	80 - 140
2	Seut Bridge	48 - 55	1.62 – 1.7		8 - 13	10 - 200		80 - 150
3	Eidsvoll	25 - 35	1.9 – 2.0	37 - 48	13 - 19	2 - 5	2 - 6	175 - 210
4	Hvalsdalen	31-39	1.86 – 1.95	40-49	9-18	5-20[3]	2-6	110 - 215
5	Skøyen - Asker	22 - 40	1.8 – 2.1		3	10 - 200		110 - 180
6	RVII	30 - 40	1.82 – 1.89	28 - 45	8 - 18	7 - 135	1.2 – 2.6	170 - 200
7	NGI car park	30 - 35	1.86 – 2.0		7 - 13	3 - 120		120 - 210
8	Danviksgata	50 - 55	1.72 – 1.78	48	30	7 – 8	1.5	100 - 170
9	Lierstranda	32 - 42	1.83 – 1.95	31 - 36	13 - 19	7 - 15	1.4 – 2.0	125 - 175
10	Hvittingfoss	22-36	1.91-2.09	17-36	4 - 6	>100		150 - 250
11	Smørgrav	35 - 45	1.80 - 1.93	36 - 60	9 - 22	5 - 77		105 - 230
12	Vålen	35 - 47	1.85 – 2.01	37 - 39	15 -22	5 - 15	1.2 – 1.8	100 - 240
13	Farriseidet 0 – 3 3 - 9	 > 400 75 - 120	 1.07 1.41-1.57			 70-140		34 - 100
14	Månejordet 2.5–5.5 5.5-14.5	 28 – 50 25 - 40	 1.83-2.09 1.83-2.00	 20 24-27	 14-16 6-9	 <10 50-350	4.5-5.5	110 - 180
15	Skienselven	26-33	1.95 – 2.0		3	110-240		86 - 150
16	Tiller	30 - 45	1.8 – 2.0	35 - 40	2 - 8	5 - 1000	2 – 4	75 – 230
17	Berg	25 - 38	1.95 – 2.05	30	7 - 10	4 - 10	5 - 3	100 - 200
18	Esp	30 - 50	1.75-1.95	30 - 40	3 - 15	10-115	2 - 4	100 - 220
20	Klett	25 - 35	1.92-1.94	30 - 35	4 - 10	10-240	1.5 – 3?	120 - 250
21	Dragvoll	30 - 42	1.88 – 2.0	28 - 48	4 - 12	16 - 152	1 - 2	110 – 190
22	Eberg	50 - 70	1.6 – 1.8	42 - 62	7 - 18	5 - 10	1 – 2	65 – 175
22	Rosten	20 - 34	2.0 – 2.23	38 – 39	4 – 10	4 - 45	5 - 9	155 – 345
23	Saupstad	24 - 30	2.0 – 2.14	32	5 – 7	42 – 150	2.5	155 – 410
24	Hoseith	20-30	2.05 – 2.18		5	11 – 138		185 - 195
25	Okstad	20-40	1.78 – 2.02			3 – 5		200 - 290
26	Rissa	28 - 40	1.85 – 2.0	42 - 47	7 - 12	10 - 60	2 - 4	100 - 280
27	Glava	30 - 35	1.8 – 2.0	30 - 60	15 - 30	7 - 10	4 - 5	125 – 230
28	Kattmarka	28 - 50	1.94 - 2.0	37 - 55	1.4 - 1.8	7 - 43	1.45 - 7.0	105 - 190
29	Bothkennar	66 - 72	1.58 – 1.61	17 - 35	42 - 53	8 - 13	2	102 - 144

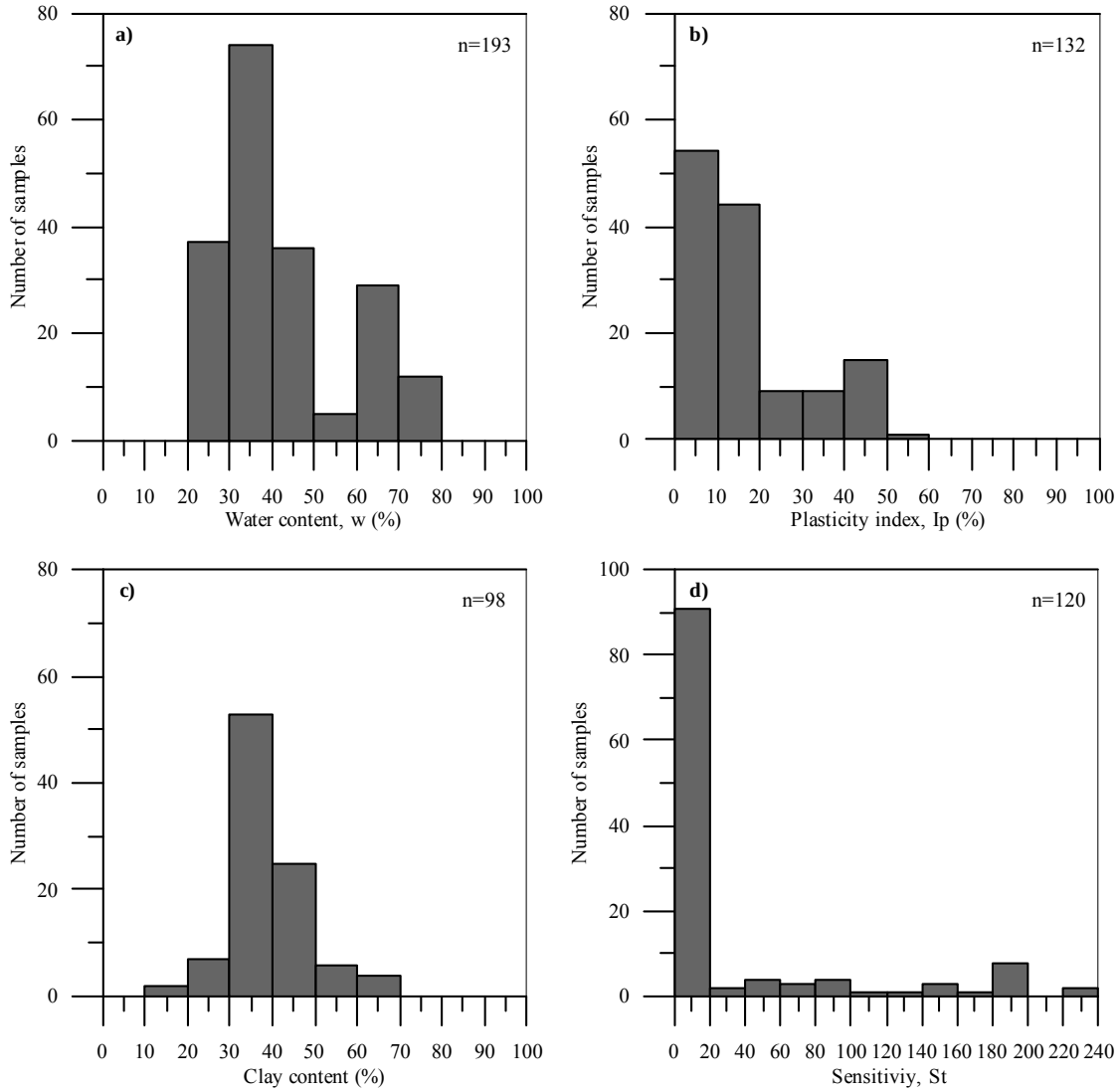

Figure 5: Histogram of a) water content, b) plasticity index, c) clay content and d) sensitivity showing content of Norwegian clay database.

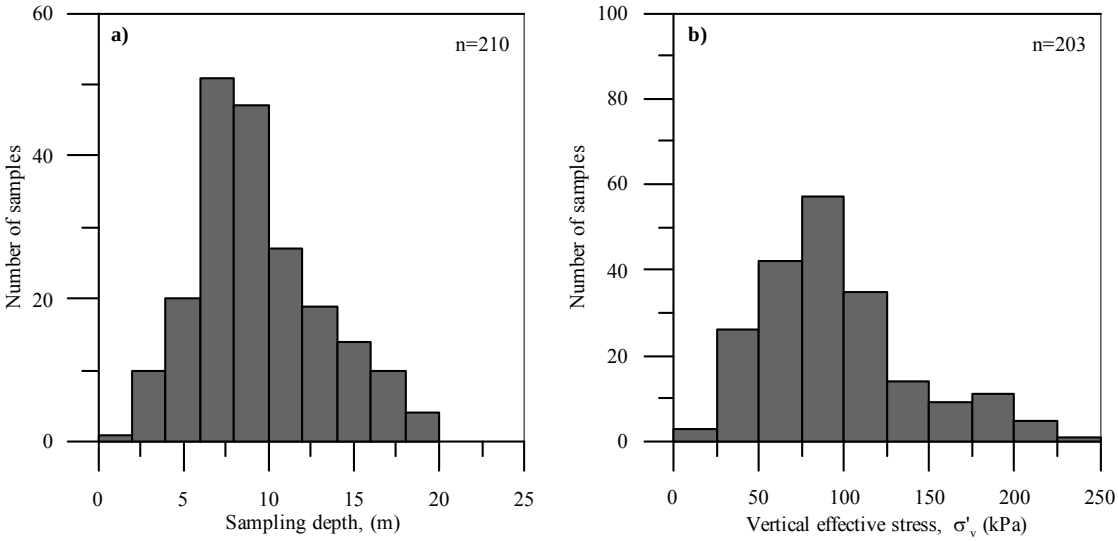

Figure 6: Histogram of a) sampling depth and b) vertical effective stress showing content of Norwegian clay database.

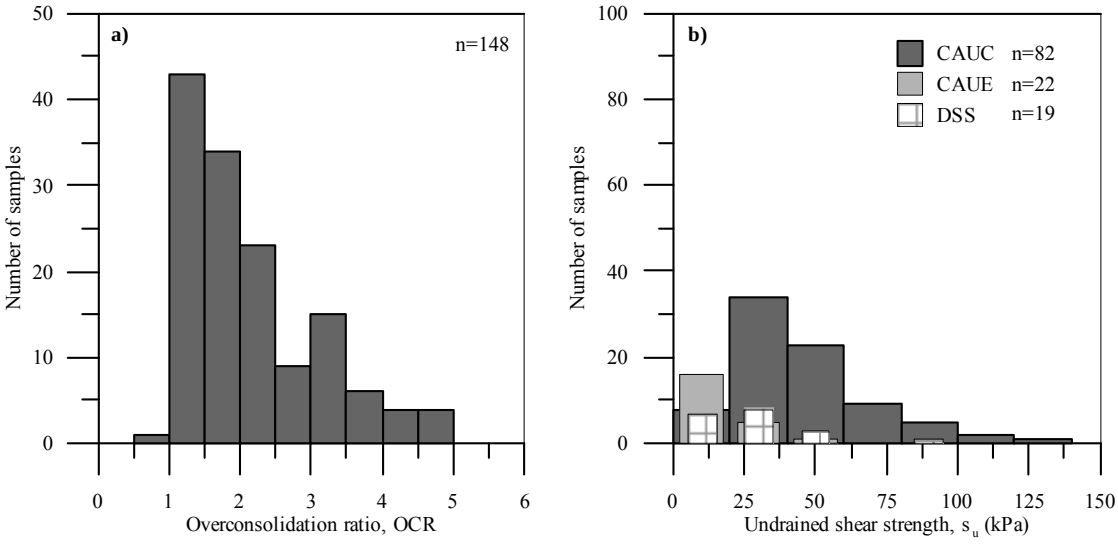

Figure 7: Histogram of a) overconsolidation ratio and b) undrained shear strength from CAUC, CAUE and DSS tests showing content of Norwegian clay database.

In addition to index properties and results from laboratory tests, the compiled database contains results from cone penetration tests (CPTU). Typically the selected CPTU data has been averaged over a 0.5 m thick zone, covering parts both above and below sampling depth. The net cone resistance (q_{net}) of the database falls within a wide range of 200 to 1800 kPa, with the highest number of observations at 300 kPa which corresponds to a shallow depth of about 5 to 6 m (Fig. 8b). The *in situ* shear wave velocity (V_s) data range between 50 and 300 m/s with the majority of the data between 125 and 200 m/s (Fig. 8a). With the exception of Onsøy and Farriseidet the data follows a very similar depth pattern (Figure 9). V_s values are typically 120 m/s at ground level and increase to 180 m/s and 200 m/s at 10 m depth and 12 m depth respectively. The very soft high water content organic clays at Onsøy and especially Farriseidet show much lower values of V_s. The Hvittingfoss data seems to show relatively high Vs values. This may be due to the somewhat coarse nature of the material.

Shear wave velocity data for the Trondheim area are shown on Figure 10. It can be seen that broadly speaking the data can be divided into two groups. The main group show similar values to those from Southern Norway with V_s increasing from about 100 m/s at ground level to 200 m/s at about 12 m depth. There is a second group of sites all located in south and south-west Trondheim (comprising the Rosten, Saupstad, Okstad and Hoseith sites) with higher values. Here the V_s values reach 300 m/s at 10 m to 12 m. The very soft clay at Dragvoll show the lowest values on average. Generally it can be seen that all the Trondheim data fits with those from South Norway except for the Rosten (high OCR at the bottom of slope), Hoseith and Okstad sites.

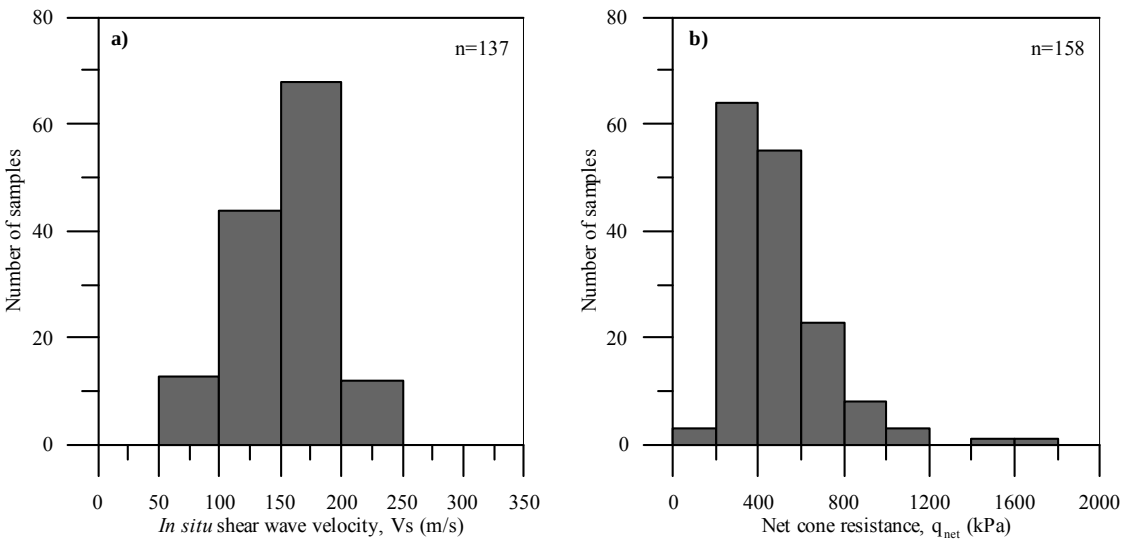

Figure 8: Histogram of a) in situ shear wave velocity and b) net cone resistance from CPTU tests showing content of Norwegian clay database.

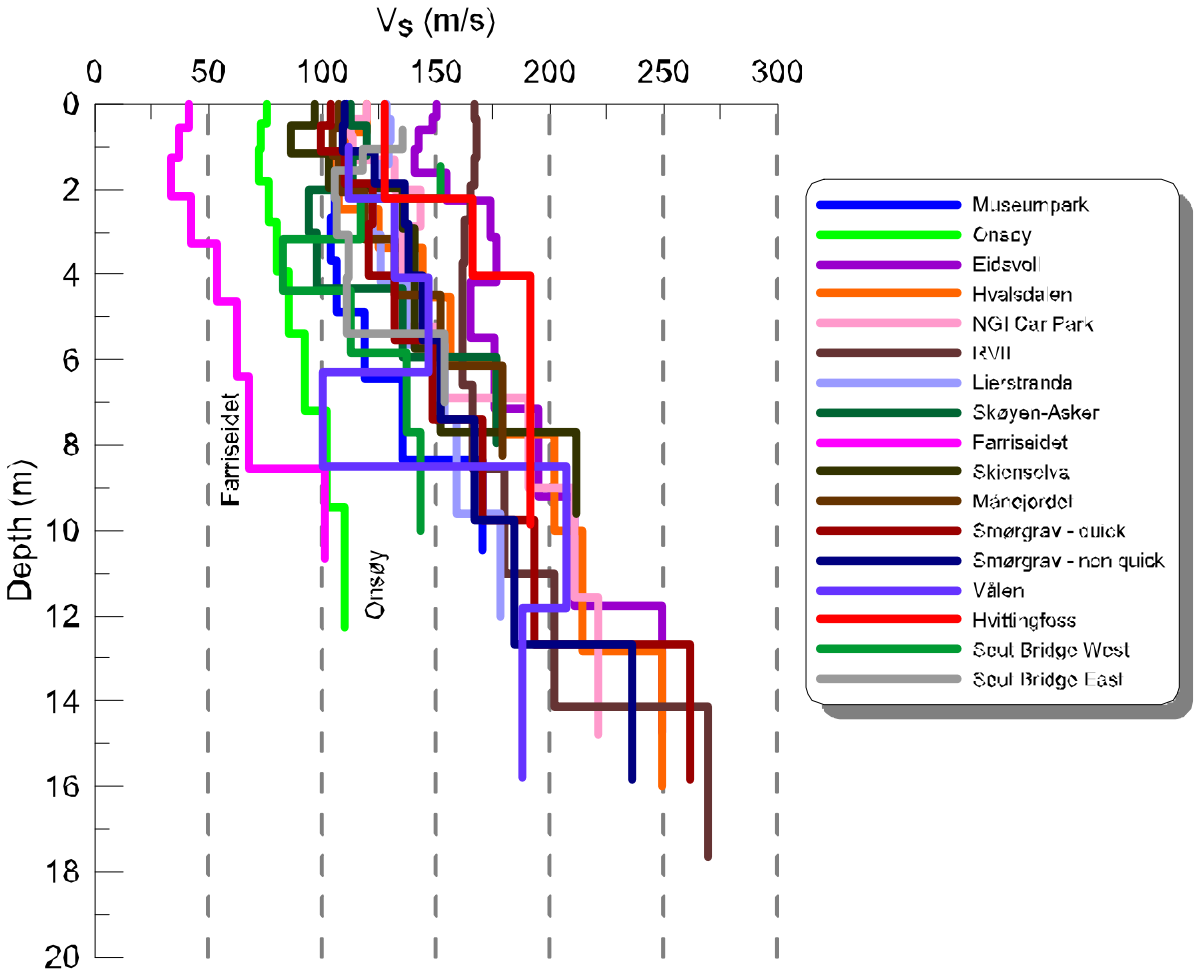

Figure 9: In situ shear wave velocity profile for sites in Southern Norway.

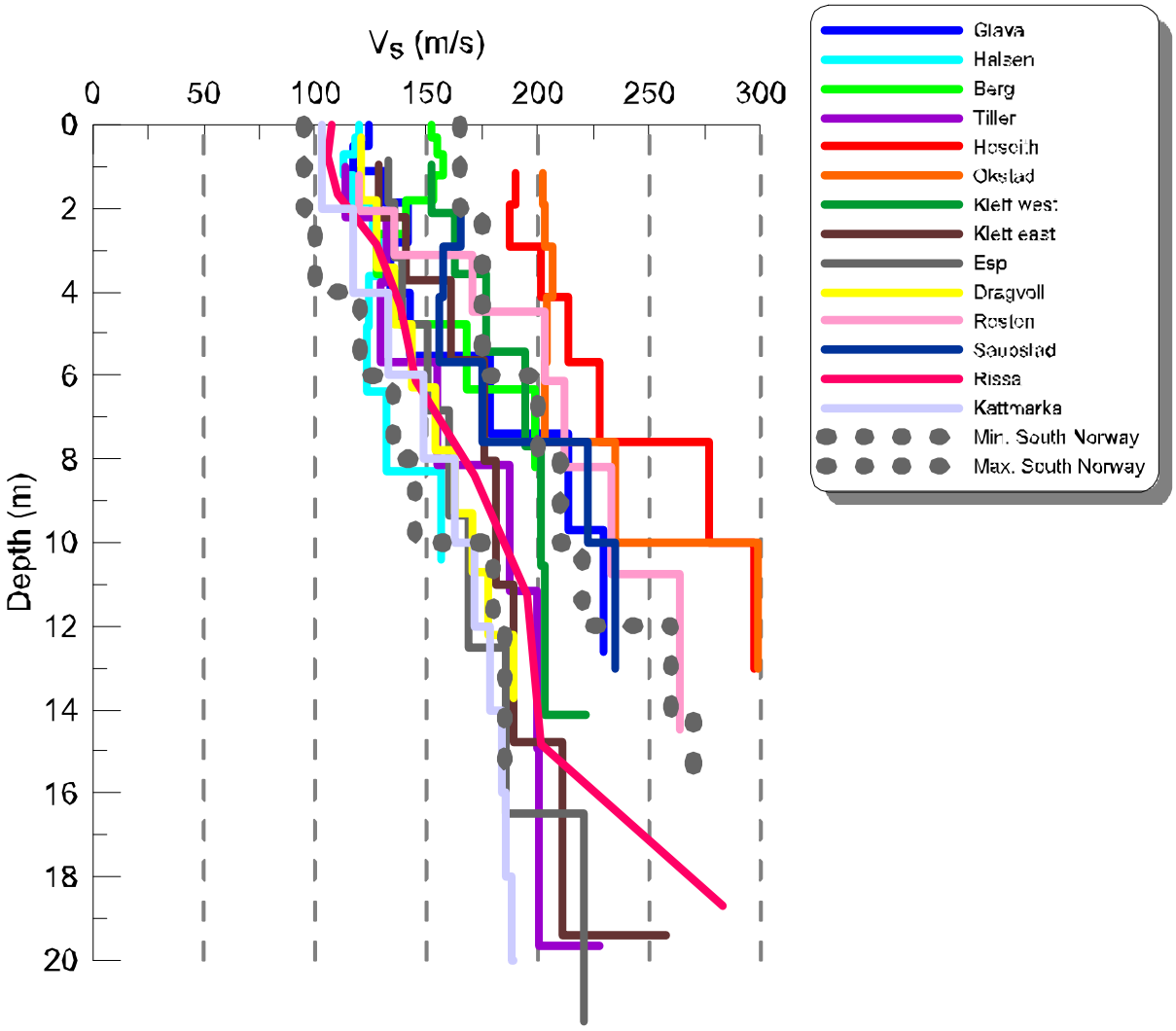

Figure 10: In situ shear wave velocity profile for sites in the Trondheim region.

4 Correlations with index properties

Correlations between index parameters and V_s or G_{max} can provide rapid estimates useful for preliminary design and for verifying *in situ* and laboratory results. (Hardin, 1978) suggested that G_{max} for clays depends on the *in situ* (or applied) stress (σ'), void ratio (e), and OCR. It has been shown, however, that the effects of OCR are, to a large extent, taken into account by the effect of void ratio and could be neglected, see for example (Leroueil and Hight, 2003). The empirical equation describing the influence of the controlling factors on G_{max} can then be written as follows:

[4] $$G_{max} = SF(e)(\sigma'_v \sigma'_h)^n p_a^{(1-2n)}$$

where S is a dimensionless parameter characterizing the considered soil; $F(e)$ is a void ratio function; σ'_v and σ'_h are the vertical and horizontal effective stresses, respectively; n is a parameter indicating the influence of stress; and p_a is the atmospheric pressure.

Figure 11 presents the relationship between in situ shear wave velocity and σ'_{v0} for all sites in the database. Results show a clear tendency for V_s to increase with σ'_{v0}. The best fit equation for the data gives a regression coefficient of 0.68.

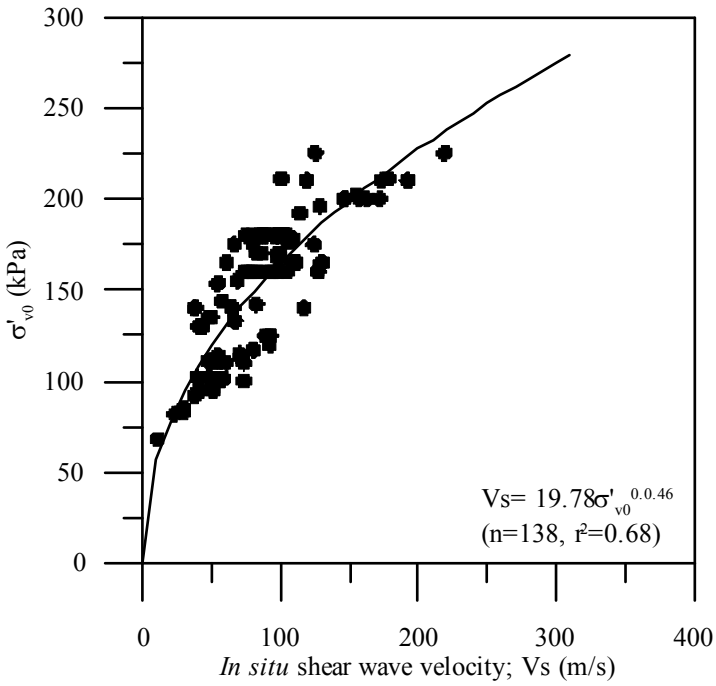

Figure 11: In situ shear wave velocity against vertical effective stress for all sites in the database.

For this work, use was made only of highest quality samples. Initially, the V_s values corresponding to sample depths were chosen and G_{max} was calculated using the sample density and Eq. [1]. The void ratio was calculated using:

[5] $$e_0 = \frac{G_s\,\gamma_w\,(1+w)}{\gamma_{tot}} - 1$$

where G_s is the specific gravity of soil solids, γ_w is the unit weight of water, w the water content, and γ_{tot} the total unit weight of the soil. G_{max} values were normalized by the corresponding *in situ* vertical effective stress (σ'_{v0}). G_{max}/σ'_{v0} typically varies between 250 and 1000 in the database. The relationship between G_{max}/σ'_{v0} and *e* is shown in Figure 12.

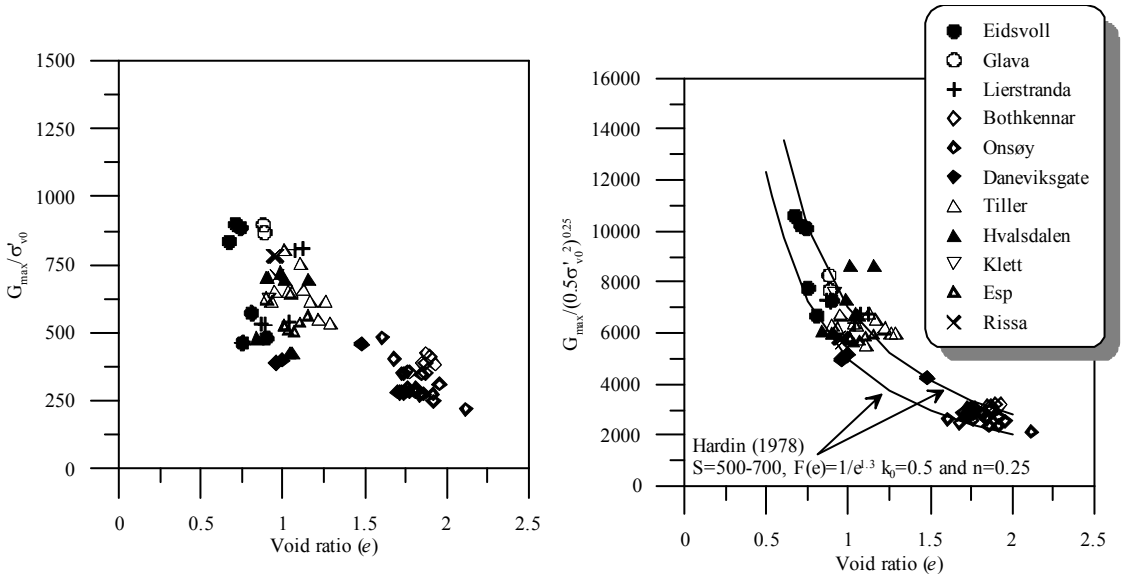

Figure 12: (Left) Relationship between G_{max}/σ'_{v0} and void ratio e. (Right) G_{max} normalized according to (Hardin, 1978) and (Hight and Leroueil, 2003) and e. Klett north and south

As expected, G_{max}/σ'_{v0} decreases with an increase in *e* in a manner similar to that described by others, see for example (Jamiolkowski et al., 1991), for a variety of soils. In Figure 12b the data have been normalized as suggested by (Hardin, 1978) and (Hight and Leroueil, 2003), as described in eq. [4]. Two lines have been added corresponding to S = 500-700, $F(e) = 1/e_{1.3}$, $K_0 = 0.5$ (where K_0 is the coefficient of earth pressure at rest), and $n = 0.25$. It can be seen that the fit is good and that S ranges from 500 to 700. This further confirms that G_{max} values for Norwegian clays are consistent with those from a large volume of other published experimental data.

The effect of void ratio on Vs is presented on Figure 13. It is observed that V_s decreases as the void ratio increases. The relationship between the void ratio and V_s takes a form similar to that proposed by (Taboada et al., 2013) for the Bay of Campeche clay:

[6] $$V_s = \frac{A}{\exp(B\,e_0)}$$

where the constant A varies from 300-757 and B is in the range 0.68-0.99.

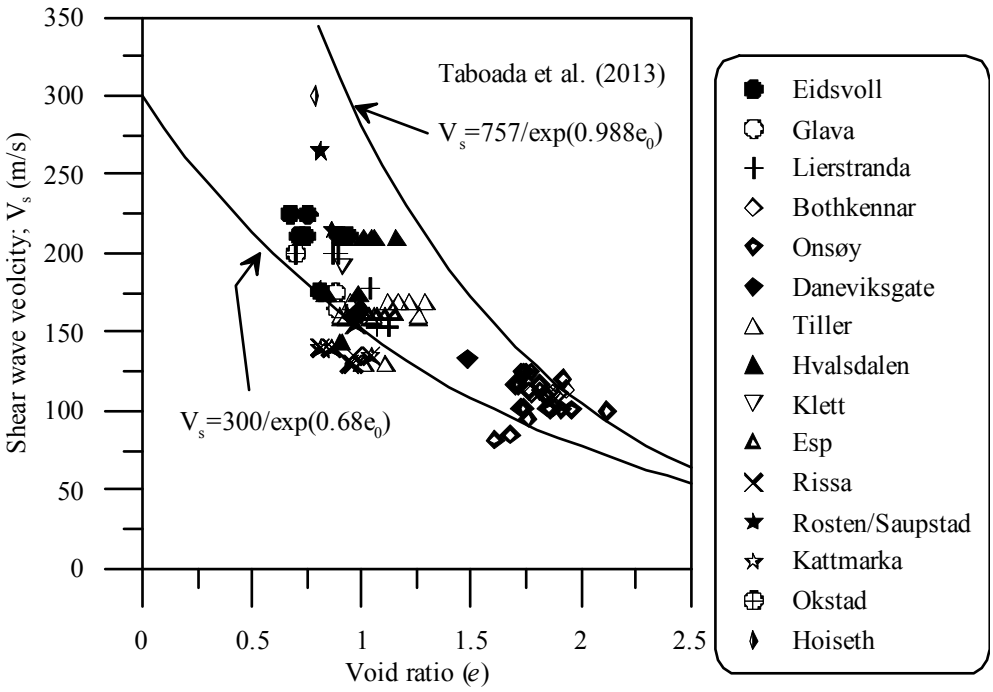

Figure 13: In situ shear wave velocity (Vs) as a function of void ratio.

Norwegian practice normalizes G_{max} with respect to the sum of consolidation stress and attraction to obtain a dimensionless parameter that depends on friction only, e.g. (Janbu, 1985). For the case of the small-strain shear modulus, (Langø, 1991) suggested that the normalized small-strain shear modulus g_{max} can be written as

[7] $$g_{max} = \frac{G_{max}}{\sigma_m' + a}$$

where σ'_m and a are the mean effective consolidation stress and the attraction measured in triaxial tests, respectively. According to (Langø, 1991), (Long and Donohue, 2007), (Long and Donohue, 2010) and (L'Heureux et al., 2013) a systematic variation in normalized shear modulus may be obtained by plotting g_{max} as a function of water content. On Figure 14, the data were normalized by σ'_{v0} and attraction was assumed to equal 3 kPa, which is a typical value for the clays under study, from (Janbu, 1985). A reasonable correlation between g_{max} and w can be seen. The trend is similar between g_{max} and I_p. Results show that g_{max} decreases with increasing water content and I_p.

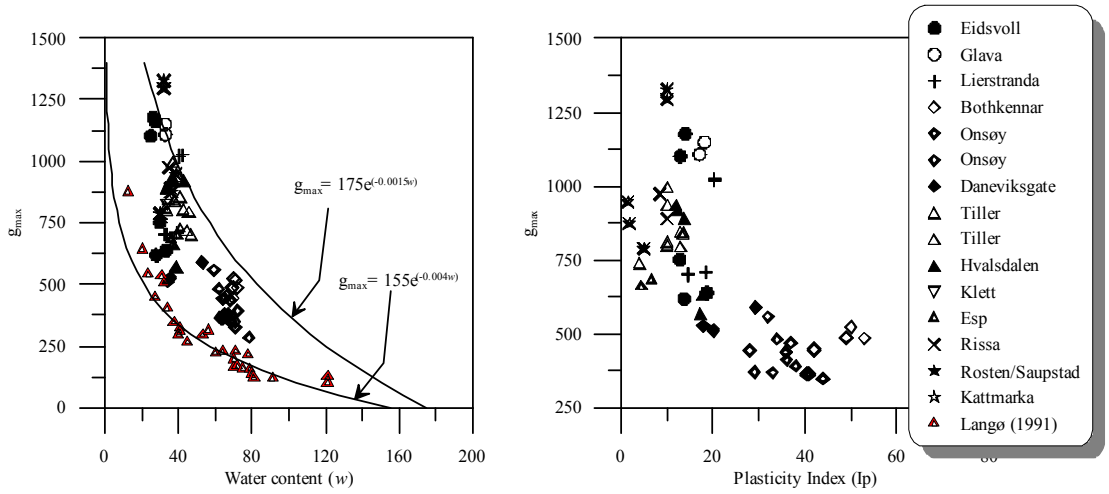

Figure 14: Normalized shear modulus g_{max} versus water content and plasticity index (Ip).

5 Correlations with cone penetration test results

The piezocone penetration test (CPTU) is a common tool used for characterization of soft and sensitive clay deposits. Various researchers have studied relationships between CPTU parameters and V_s in clayey soils (Table 5). These studies have explored relationships between in situ V_s and various parameters such as CPTU tip resistance (q_c), corrected tip resistance (q_t), cone net resistance (q_{net}), sleeve friction (f_s), pore pressure parameter (B_q), normalized cone resistance (Q_t), effective stress (σ'_v) and void ratio (e).

Table 5: Example of studies published between cone penetration test results and in situ Vs for fine grained soils.

Study/Reference	Location	no. of sites	Material	Method of Vs measurement
(Tanaka et al., 1994)	Japan	–	Clay	–
(Hegazy and Mayne, 1995)	Worldwide	61	Glacial, marine, deltaic	CHT, Downhole, SCPT, SASW
(Mayne and Rix, 1995)	Worldwide	31	Glacial, marine, deltaic	CHT, Downhole, SCPT, SASW
(Piratheepan, 2002)	California, Japan, Canada	–	Fine and coarse-grained	–
(Mayne, 2006)	Worldwide	–	Glacial, marine, deltaic	CHT, Downhole, SCPT, SASW
(Long and Donohue, 2010)	Norway	11	Soft clay	SCPT, MASW, SASW, CHT
(Taboada et al., 2013)	Gulf of Mexico	–	Clay	P-S suspension logging system

An overview of the V_s prediction equations found in the literature for clays is presented in Table 6. For consistency, some of the equations have been modified to use of SI units: q_c, q_t, q_{net}, f_s and (σ'_v) are in kPa and depth (D) is in meters. The number of points used to develop each correlation equation is presented as well as the coefficient of determination (r^2).

Based on work at NGI (e.g. Powell and Lunne, 2005) and on experience in Ireland and the UK Long (2008) suggested that CPTU sleeve friction (f_s) measurements are less reliable than cone resistance (q_t), which in turn are less reliable than pore pressure (u_2). It follows then that the most reliable correlations between Vs and CPTU parameters are likely to involve q_t or u_2 and that use of f_s readings could be unreliable.

Table 6: Example of available CPTU-Vs correlations for clays

Study/Reference	Number of data pairs	r^2	Vs (m/s) or Gmax (kPa)
(Tanaka et al., 1994)			$G_{max} = 50 \cdot (q_t - \sigma_{v0})$
(Hegazy and Mayne, 1995)	406	0.890	$V_s = 14.13 \cdot (q_c)^{0.359} \cdot (e_0)^{-0.473}$
(Hegazy and Mayne, 1995)	229	0.780	$V_s = 3.18 \cdot (q_c)^{0.549} \cdot (f_s)^{0.025}$
(Mayne and Rix, 1995)	339	0.830	$V_s = 9.44 \cdot (q_c)^{0.435} \cdot (e_0)^{-0.532}$
(Mayne and Rix, 1995)	481	0.740	$V_s = 1.75 \cdot (q_c)^{0.627}$
(Piratheepan, 2002)	20	0.910	$V_s = 11.9 \cdot (q_c)^{0.269} \cdot (f_s)^{0.108} \cdot D^{0.127}$
(Mayne, 2006)	161	0.820	$V_s = 118.8\log(f_s) + 18.5$
(Long and Donohue, 2010)	35	0.613	$V_s = 2.944 \cdot (q_t)^{0.613}$
(Long and Donohue, 2010)	35	0.758	$V_s = 65 \cdot (q_t)^{0.15} \cdot (e_0)^{-0.714}$
(Long and Donohue, 2010)		0.777	$V_s = 1.961 \cdot (q_t)^{0.579} \cdot (1 + B_q)^{1.202}$
(Taboada et al., 2013)	274	0.94	$V_s = 14.4 \cdot (q_{net})^{0.265} \cdot (\sigma'_{v0})^{0.137}$
(Taboada et al., 2013)	274	0.948	$V_s = 16.3 \cdot (q_{net})^{0.209} \cdot \left(\frac{\sigma'_{v0}}{w}\right)^{0.165}$

Multiple regression analyses were conducted on the Norwegian clay database to provide power function expressions for *in situ* Vs in terms of q_{net}, obtained from cone penetration tests and soil properties. The relationship with the highest coefficient of correlation using q_{net}, and one additional parameter was:

[8] $$V_s = 8.35 \cdot (q_{net})^{0.22} \cdot (\sigma'_{v0})^{0.357}$$

The coefficient of determination r^2 is 0.73 and a total of 115 datasets were used in the analysis. The trend between the *in situ* measured V_s and the prediction given by equation [8] is illustrated in Figure 15a. The figure shows most of the predicted values of V_s are within 20 % of the measured V_s.

The prediction given by equation [8] was improved when the water content was introduced giving rise to the following expression:

[9] $$V_s = 71.7 \cdot (q_{net})^{0.09} \cdot \left(\frac{\sigma'_{v0}}{w}\right)^{0.33}$$

The coefficient of determination r^2 is 0.89 and again a total of 101 datasets were used in the analyses. The trend between in situ measured V_s and the expression given in equation [9] is presented in Figure 15b. When using equation [9] most of the predicted values of V_s are within 10-15 % of the measured V_s.

Equations 8 and 9 are similar to those presented by Taboada et al. (2013) for clays from the Gulf of Mexico (see Table 6). However, the empirical factors vary greatly. *In situ* V_s for Norwegian clays seem to be more strongly controlled by water content and vertical effective stresses, and to a lesser extent by the net cone resistance.

Application of the most prominent CPTU-based empirical equations are presented for several of the Norwegian clay sites in Appendix B. All 3 CPTU relationships (i.e. based on $q_t - e_0$, $q_t - B_q$, and $q_{net} - \sigma_{v0}' - w$) tested at the different site locations give good prediction of Vs. In general, we observe that the relationship based on q_t and e_0 will not capture the increase in V_s with depth. Such an increase is better captured with the $q_t - B_q$, and $q_{net} - \sigma_{v0}' - w$ relationships. Other observations from Appendix B is that the $q_t - B_q$ relationship will sometimes overestimate V_s, whereas that based on $q_{net} - \sigma_{v0}' - w$ will sometimes underestimate V_s.

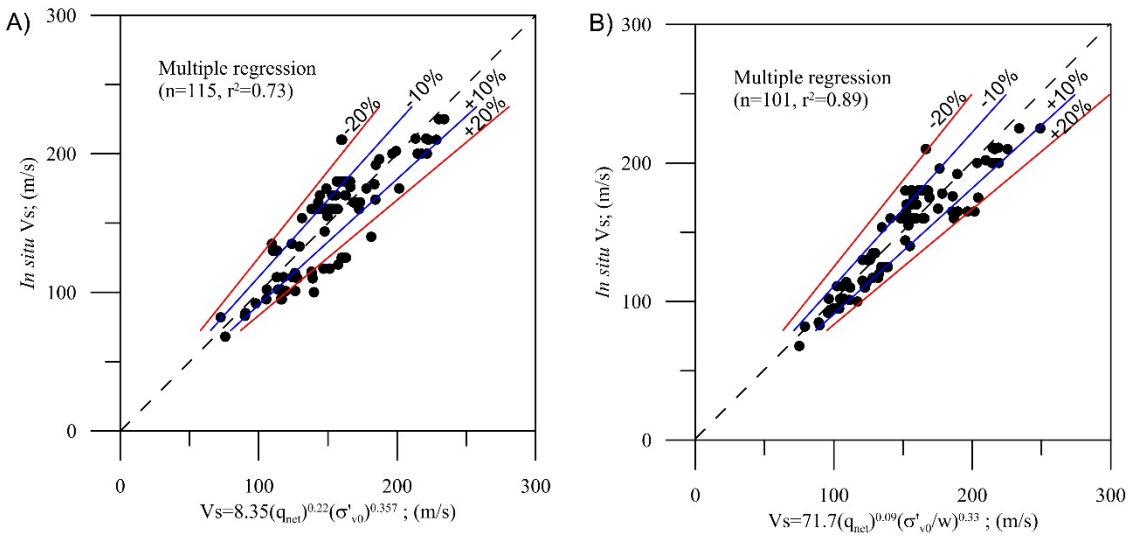

Figure 15: Comparison of measured and predicted V_s as a function of a) net cone resistance (q_{net}) and effective stress (σ'_{v0}), and b) net cone resistance (q_{net}) and effective stress (σ'_{v0}) normalized by water content (w).

6 Correlations with undrained shear strength

As discussed in chapter 1, G_{max} and V_s of cohesive soils primarily depend on void ratio, effective stress, and stress history. Similar to penetration-based correlations, relationships between V_s and undrained shear strength (s_u) for clays can be made since both properties depend on common parameters.

Several relationships have previously been proposed in the literature between V_s and s_u. For the San Francisco Bay area, (Dickenson, 1994) proposed a relationship of the form:

[10] $$V_s = 23\, s_u^{0.475}$$

where Vs is in units of m/s and s_u is in kPa and measured using the fall cone technique. (Taboada et al., 2013) used a similar approach to establish a correlation between *in situ* shear wave velocity and undrained shear strength determined from unconsolidated-undrained (UU) triaxial tests and in situ vane tests for the Bay of Campeche clay. Results from simple regression analysis obtained by (Taboada et al., 2013) take the form:

[11] $$V_s = 31\, s_u^{0.414}$$

The undrained shear strength values obtained from CAUC triaxial tests on Norwegian clay samples are plotted against in situ shear wave velocity in Figure 16A. The results show an increase in s_u (CAUC) with increasing V_s. The best fit is given by Eq. 12 with a regression coefficient of 0.83.

A similar tendency is observed for undrained shear strength obtained from extension tests (CAUE) (Fig. 16B) with the best fit given by Eq. 13. The latter has a regression coefficient of 0.61. For both s_u (CAUC) and s_u (CAUE) the scatter in the data increases for increasing Vs and the greatest variation is for the highly overconsolidated Eidsvoll and Hvalsdalen clays.

A similar plot for V_s against undrained shear strength from direct simple shear tests (DSS) is presented in Figure 17. The best relationship is given by Eq. [14] with a regression coefficient of 0.91. The data in Figure 17 is compared to the relationships proposed by Andersen (2004) (i.e. $G_{max}/s_{u,DSS}$= 800 – 900). Note that to compare with the relationships proposed by (Andersen, 2004) we made use of [Eq. 1] by varying the density between 1.6 and 1.9 Mg/m^3 and the empirical factor between 800 and 900. Figure 17 shows the 2 extreme lines from the Andersen (2004) relationship. The fit is good at low Vs value, but large difference arise for higher Vs results. The reason for these differences may come from the fact that the relationships proposed by (Andersen, 2004) are based on laboratory measurements of V_s and G_{max}, whereas *in situ* V_s data are used in this study.

Based on the results presented in Figs. 16 and 17 we propose using the following relationships for estimating *in situ* V_s from CAUC, CAUE and DSS results in Norwegian clays:

[12] $V_s = 11.12\, s_{u,CAUC}^{0.70}$ or $s_{u,CAUC} = 0.032\, V_s^{1.42}$

[13] $V_s = 8.04\, s_{u,CAUE}^{1.06}$ or $s_{u,CAUE} = 0.14\, V_s^{0.94}$

[14] $V_s = 14.87\, s_{u,DSS}^{0.69}$ or $s_{u,DSS} = 0.02\, V_s^{1.45}$

Equations 12-14 can also be used to assess undrained shear strength from Vs measurements by rewriting the relationships and solving for s_u as presented above.

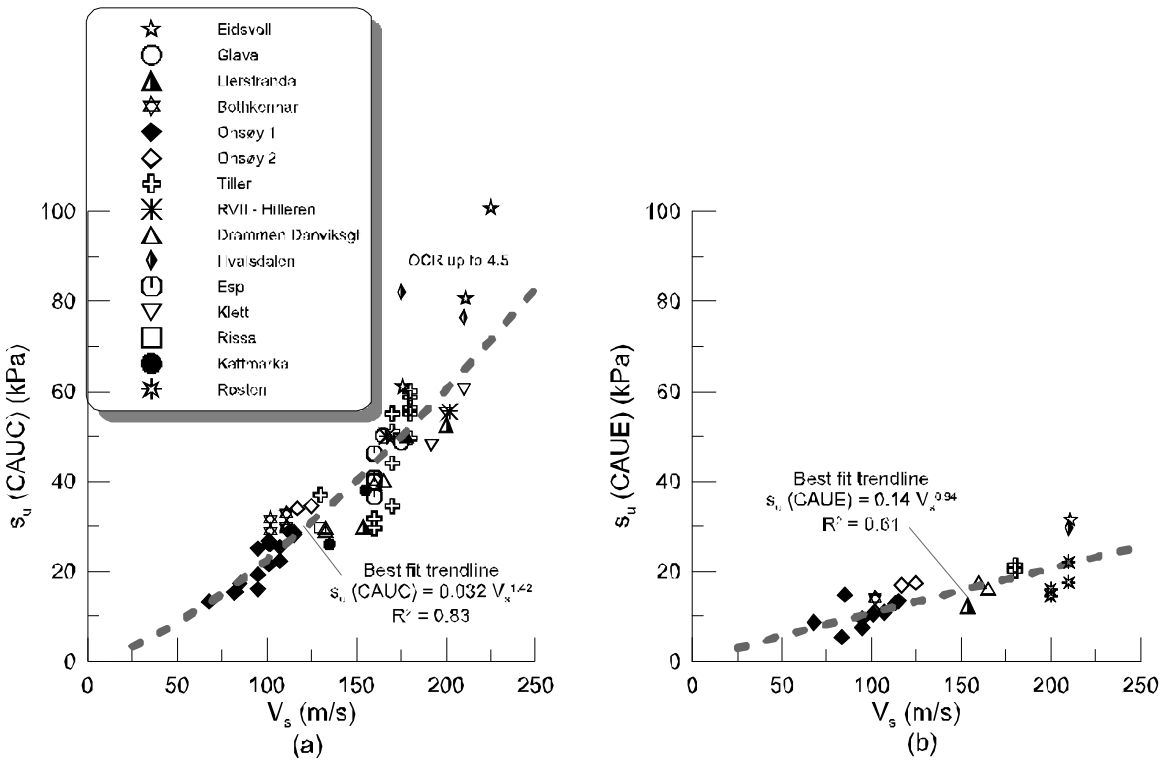

Figure 16: Results of in situ shear wave velocity against undrained shear strength from A) CAUC triaxial tests and B) CAUE triaxial tests.

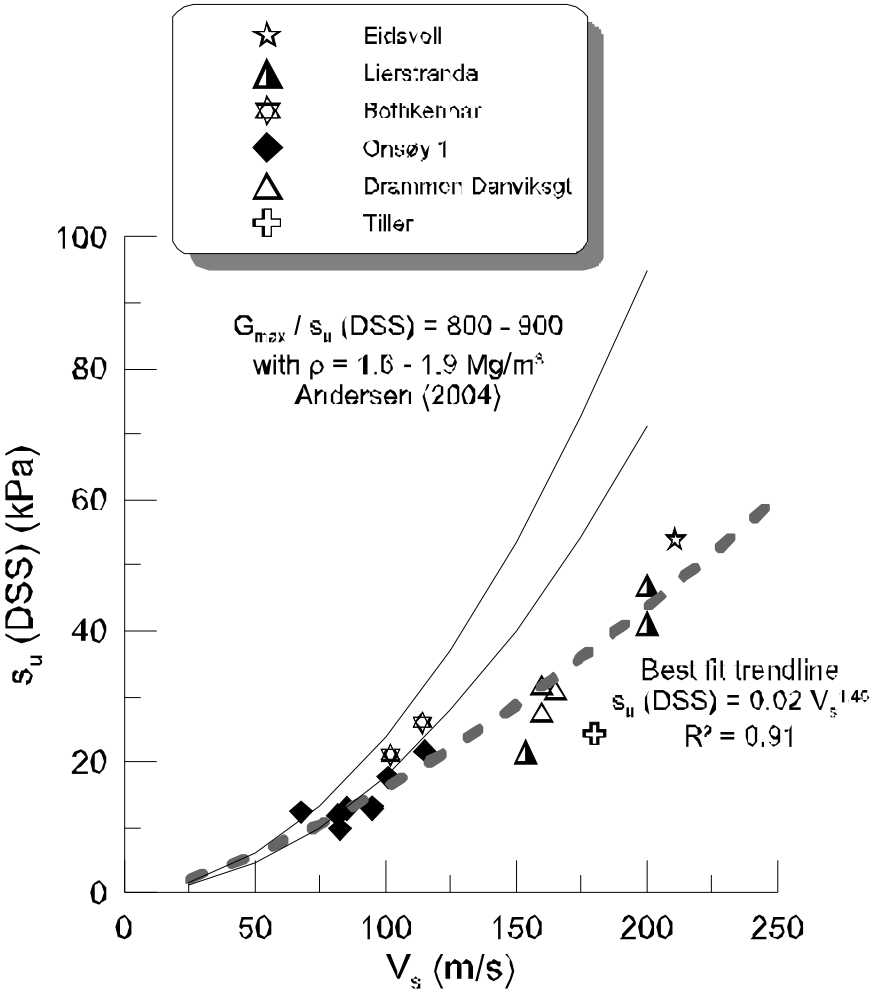

Figure 17: Results of in situ shear wave velocity against undrained shear strength from direct simple shear tests (s_{uDSS}).

7 Correlations with 1D compression parameters

In this section in situ shear wave velocity measurements are compared to the classical 1D compression parameters published by (Janbu, 1963) and (Janbu, 1969). The classical Janbu plot of 1D compression stiffness against stress is shown on Figure 18 below.

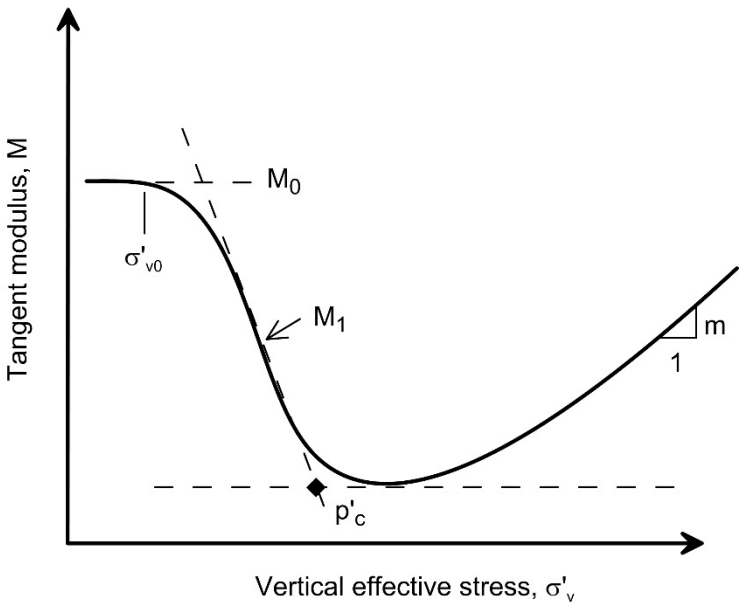

Figure 18: Classical Janbu tangent modulus versus stress model

(Janbu, 1963) used the resistance concept to interpret one dimensional consolidation in an oedometer test. He defined the tangent modulus (or the constrained modulus), M, as the ratio of the change in stress ($\delta\sigma'$) to the change in strain ($\delta\varepsilon$) for a particular load in increment, i.e.:

[15] $$M = \frac{d\sigma`}{d\varepsilon}$$

For a low stress level, around the in situ vertical effective stress (σ_{v0}'), the resistance against deformation (M_0) is large. When the stress increases this high resistance decreases appreciably owing to partial collapse of the grain skeleton. Resistance reaches a minimum (M_n) around the preconsolidation stress (p_c'). Subsequently when the effective stress is increased beyond p_c' the resistance increases linearly with increasing effective stress. In the overconsolidated range M_1 (the average between M_0 and M_n) is often used in design.

Behaviour in the normal consolidation stress range can be approximated by a linear oedometer modulus M. Hence, for $\sigma' > p_c'$:

[16] $$M = m(\sigma` - \sigma_r`)$$

where m is the modulus number and σ_r' is the intercept on the σ' axis and is the reference stress.

Correlations have been developed between 1D compression parameters obtained from oedometer tests and shear wave velocity measurements made at the various sites presented in Figure 4. Oedometer test data was obtained from tests on high quality Sherbrooke block samples or miniblock samples only was used.

The relationship between M_0 and M_1 and V_s is shown on Figure 19 below. Reasonable correlations would be expected here as V_s is a function of the current state of stress. Both M_0 and M_1 increase with increasing V_s as expected. The scatter in the data increases for increasing V_s and the greatest variation is for the highly overconsolidated Eidsvoll and Hvalsdalen clay. The best fit power trend lines shown give a reasonable R^2 values for both M_0 and M_1.

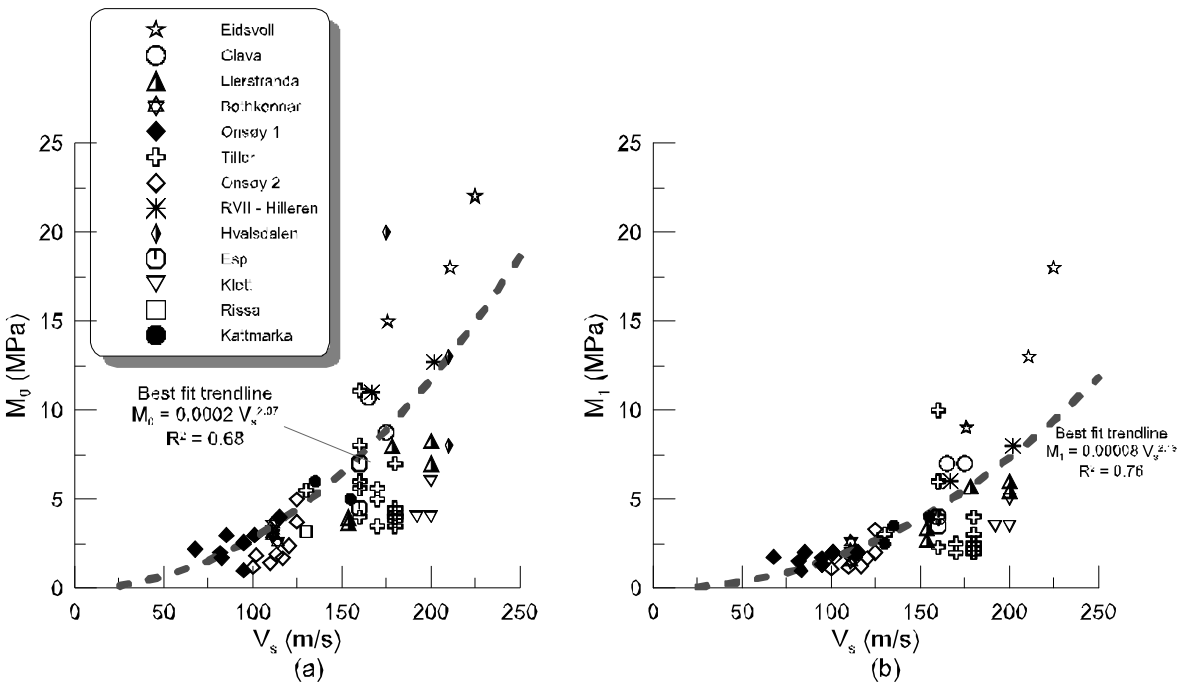

Figure 19: (a) M_0 and (b) M_1 versus V_s

Values of the preconsolidation stress (p_c' as determined by the Janbu procedure) are plotted against V_s on Fig 20A. Again a reasonably good correlation would be expected here as the shear wave velocity is strongly dependent on the maximum past stress

experienced by the clay. The relationship between p_c' and V_s is very good and the best fit power function has a r^2 value of 0.81. However, the fit is not good for OCR.

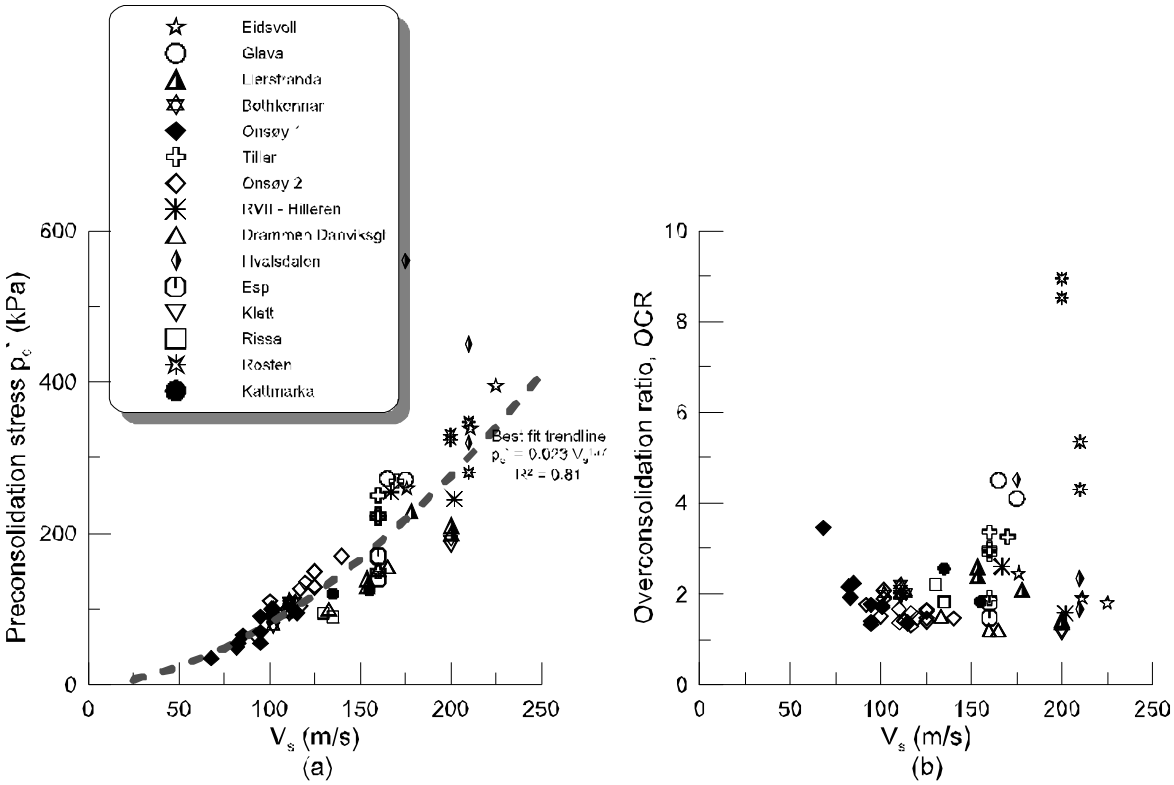

Figure 20: A) Preconsolidation pressure (p'_c) and B) Overconsolidation ratio (OCR) versus V_s.

The variation in the modulus number *m* versus shear wave velocity is shown on Fig. 21. There is a clear tendency for an increase in *m* with increasing V_s. However the fit is not as good for M_0, M_1 and p_c'. This is not surprising as you would expect V_s to represent the current state of stress not at some arbitrary higher stress stiffness.

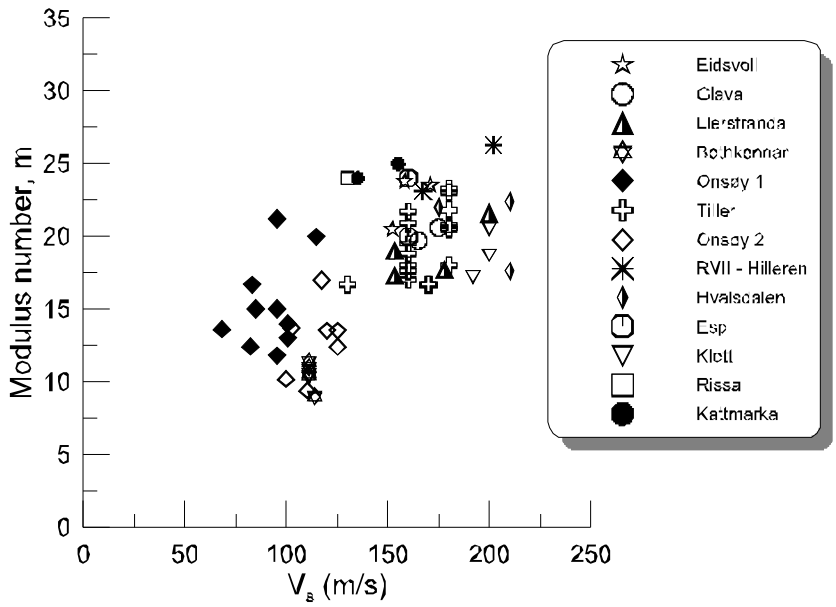

Figure 21: Modulus number versus V_s

8 Conclusions

The purpose of this study is to present guidelines and correlations to assists geotechnical engineers in estimating V_s profiles in Norwegian clays in the absence of site-specific data. For this, a database of *in situ* V_s measurements and standard geotechnical engineering material properties for Norwegian clays has been established. The database allowed the development of several empirical correlations between in situ V_s and basic soil properties, cone penetration parameters, undrained shear strength and 1D compression parameters. Based on the results from regression analyses, we recommend the use of empirical functions based on cone penetrometer data to determine the best estimate *in situ* V_s of Norwegian clay when *in situ* measurements of V_s at the site are not available. Relationships based on undrained shear strength from CAUC or DSS tests can also be used in practice. Note that the relationships presented herein can be used either to evaluate V_s from a given soil property, or the way around to evaluate soil properties from V_s.

In general, it is recommended that engineers consider all available data including available relationships, *in situ* measured V_s profiles, and site-specific geotechnical data. The use of correlations in geotechnical engineering should be limited to the conditions for which they were developed and calibrated. The recommendations presented in this report should be used in conjunction with the engineer's own experience and engineering judgment. Site-specific correlations may be developed based on a limited number of site-specific V_s measurements and using a similar functional form.

If the resulting V_s values differ from each other by more than 30%, consideration should be given to performing *in situ* shear wave measurements, or a range of V_s should be considered for design.

9 Acknowledgements

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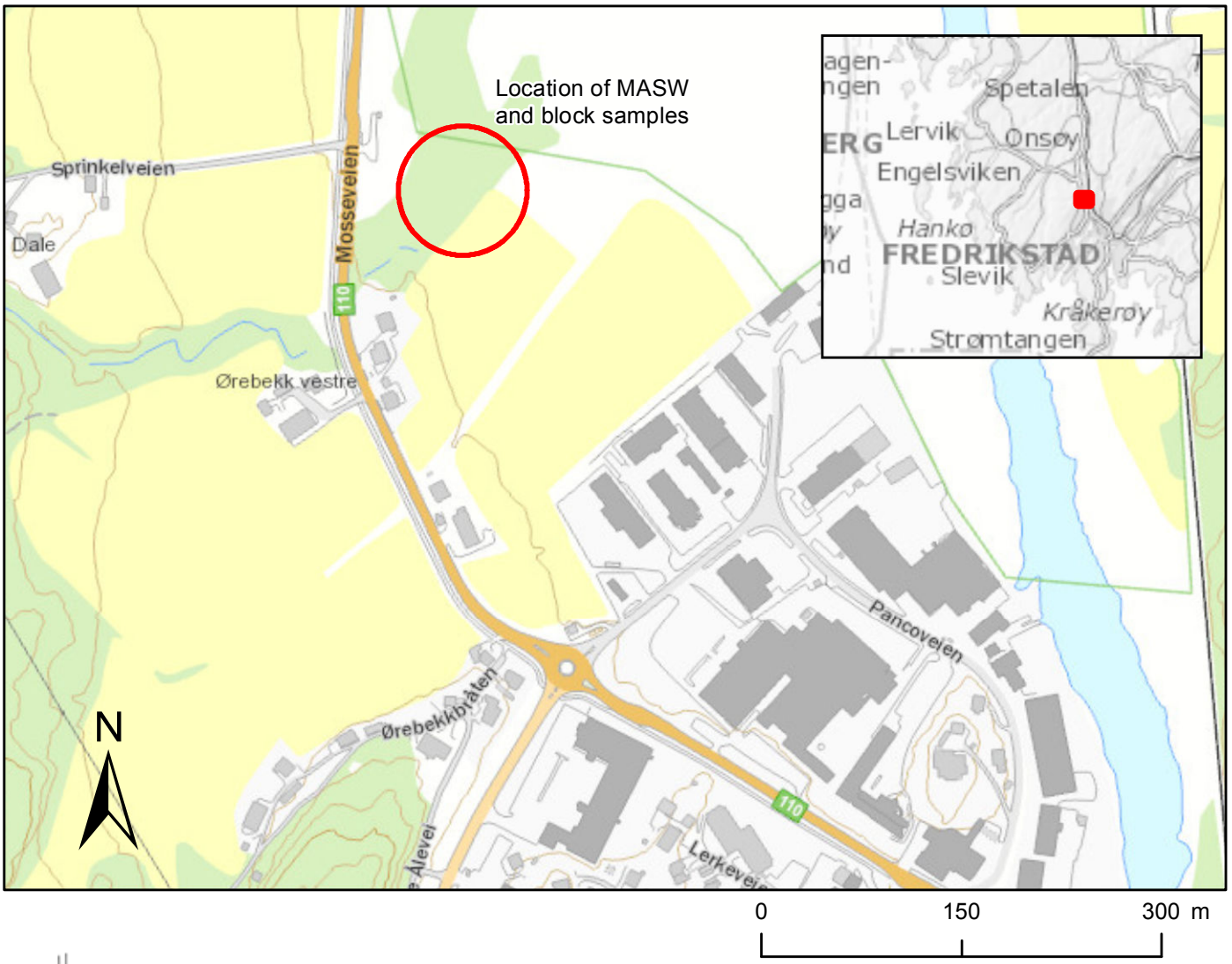
Appendix A

SITE LOCATIONS



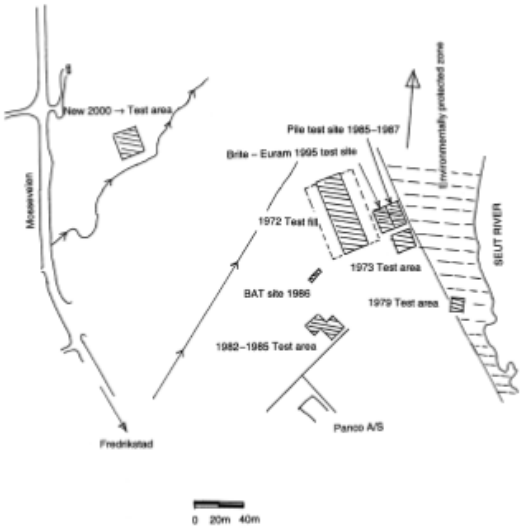
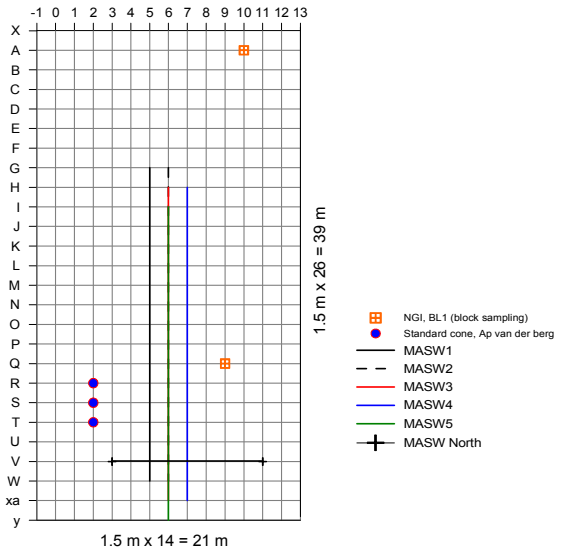


Figure 2. Layout of Onsøy test site.



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Onsøy forsøksfelt		
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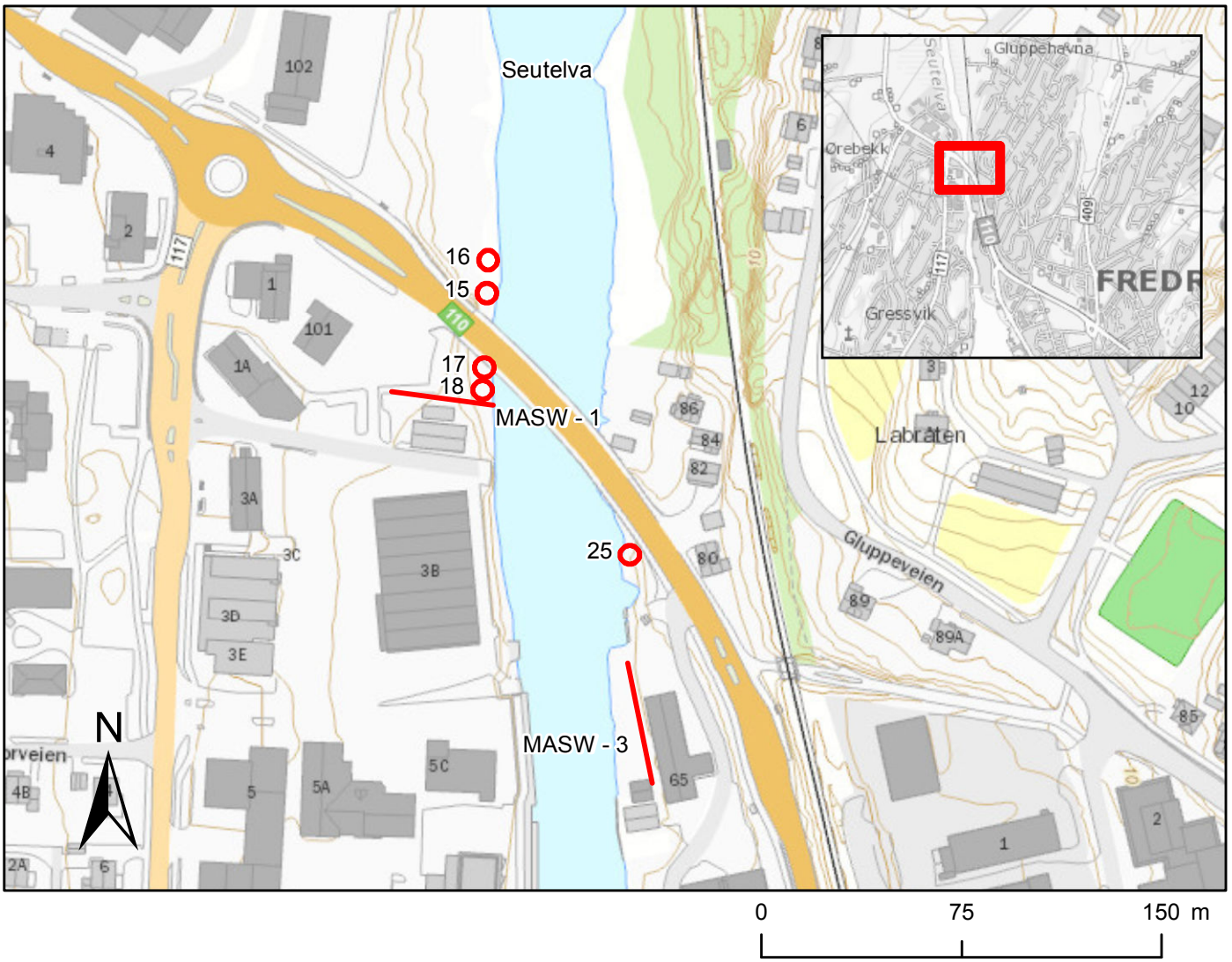
	NGI	



Målestokk (A4): 1:2 500 Kartprojeksjon: UTM_Zone_33N

Seutelva forsøksfelt		
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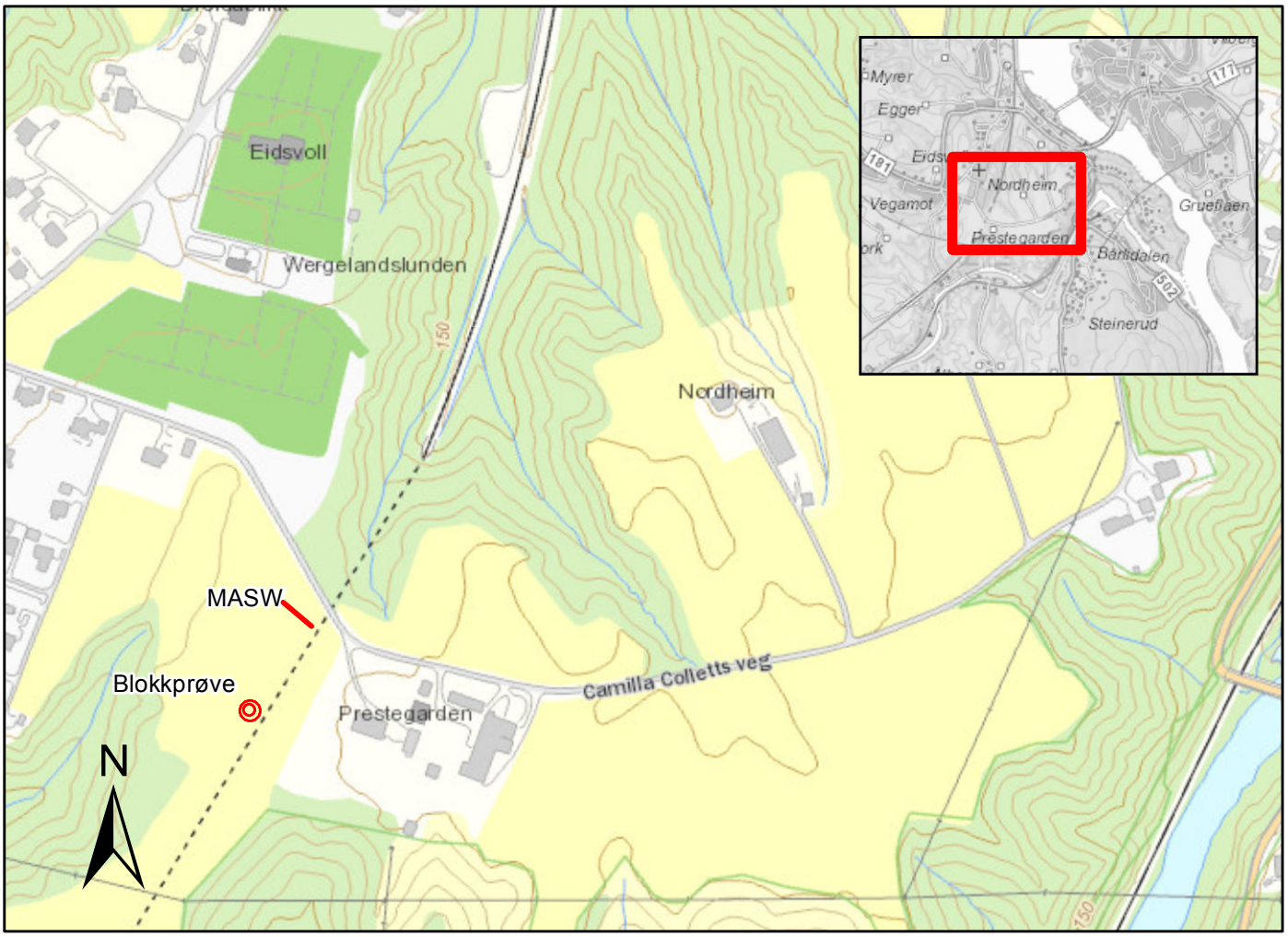
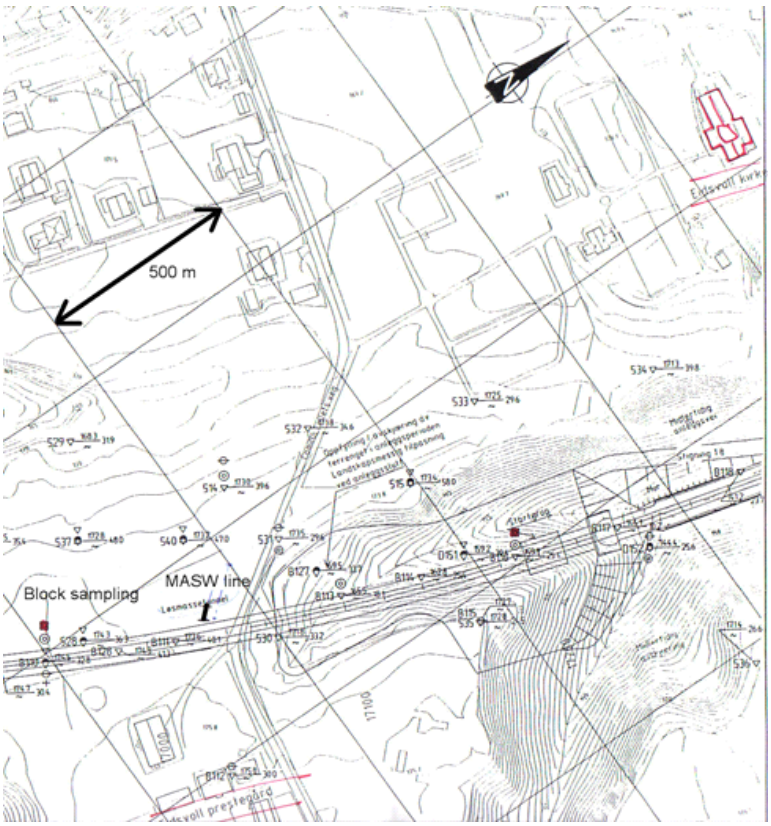
	Kontrollert	Godkjent
	NGI	





Målestokk (A4): 1:5 000 Kartprojeksjon: UTM_Zone_33N

Eidsvoll forsøksfelt		
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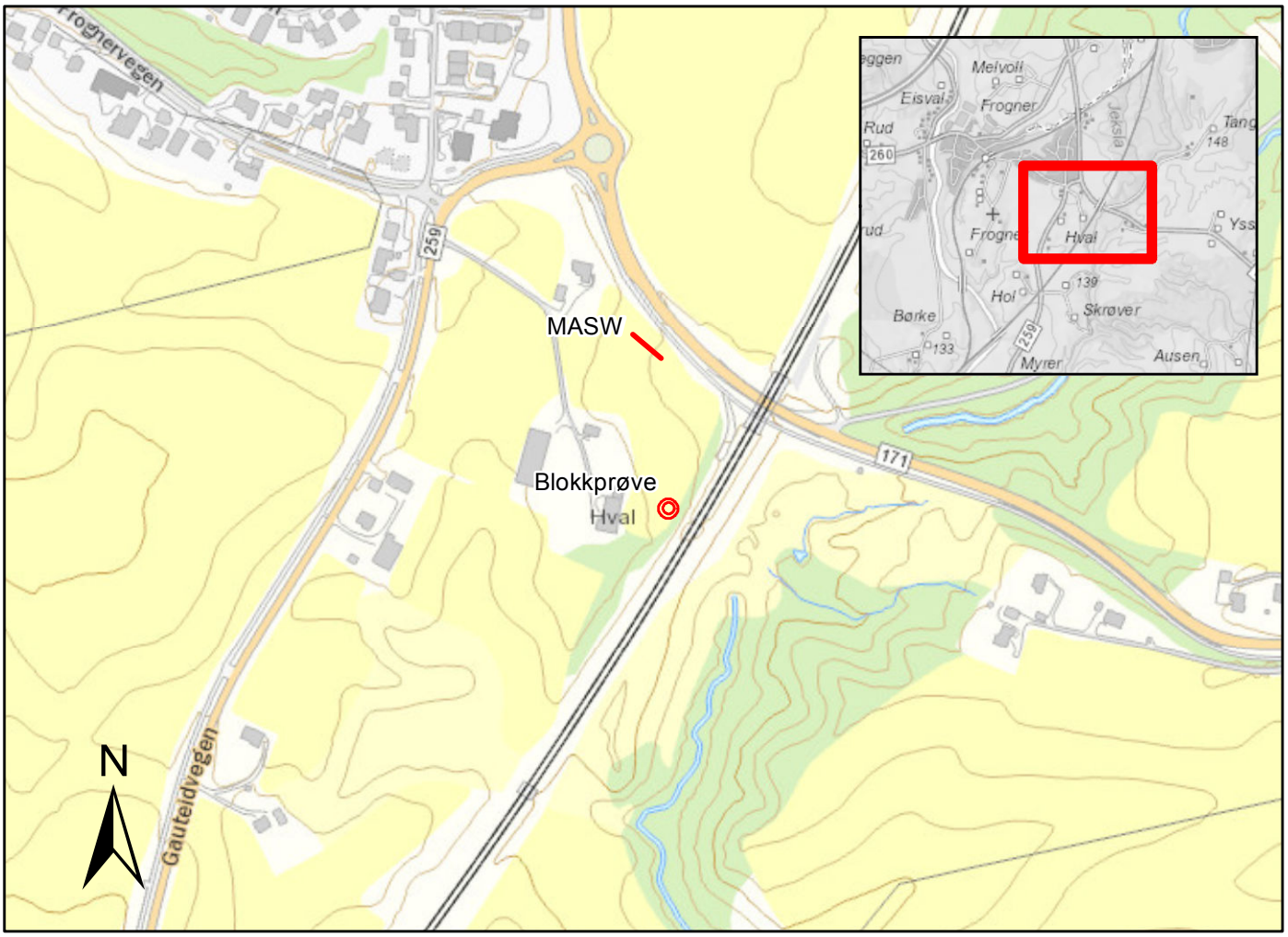
	Utført **JSL**	Dato **2015-02-24**
	Kontrollert	Godkjent
	NGI	



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Målestokk (A4): 1:5 000 Kartprojeksjon: UTM_Zone_33N

Hvalsdalen forsøksfelt		
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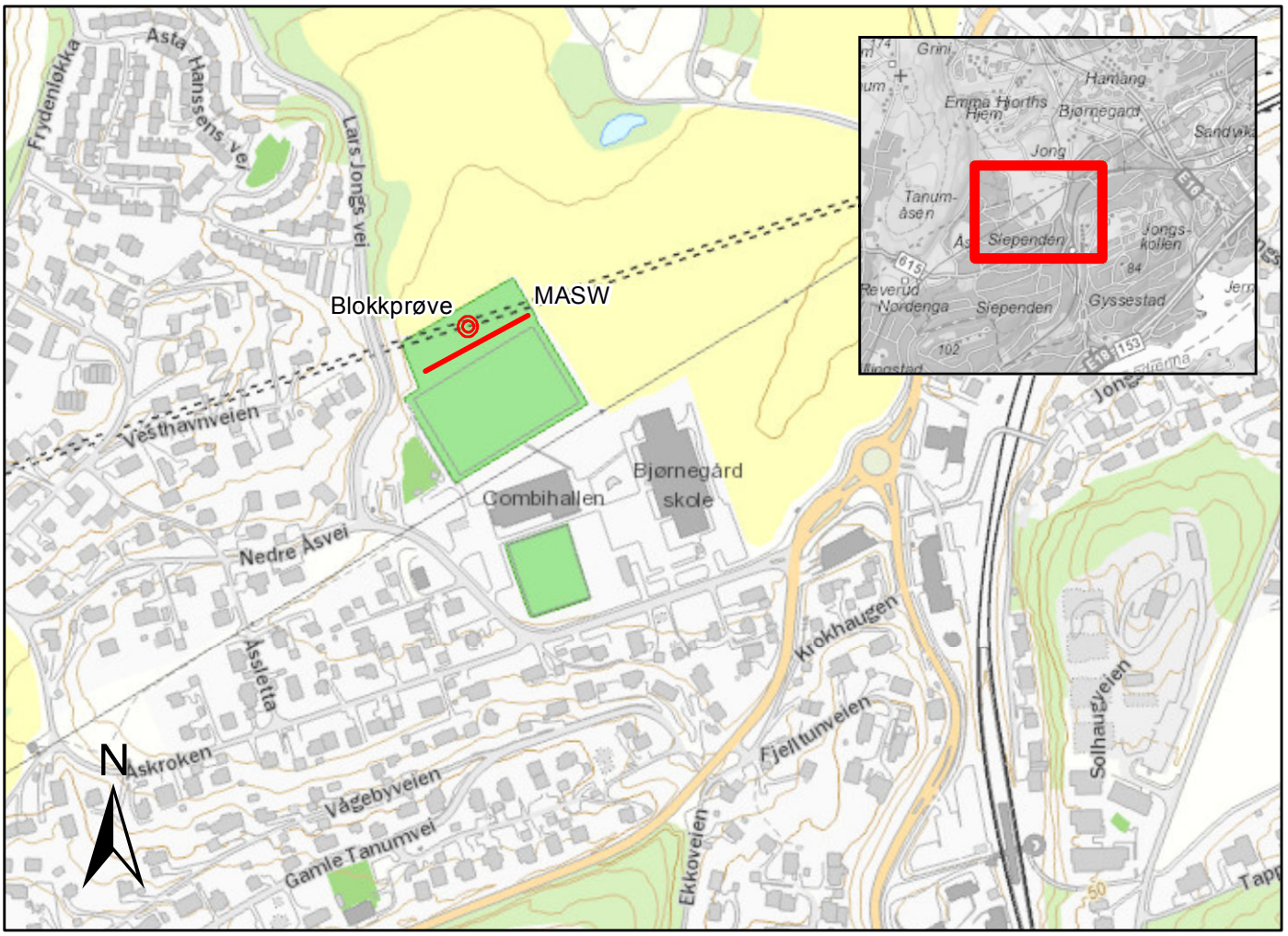
	Utført **JSL**	Dato **2015-02-24**
	Kontrollert	Godkjent
	NGI	



Målestokk (A4): 1:5 000 Kartprojeksjon: UTM_Zone_33N

Skøyen-Asker forsøksfelt		
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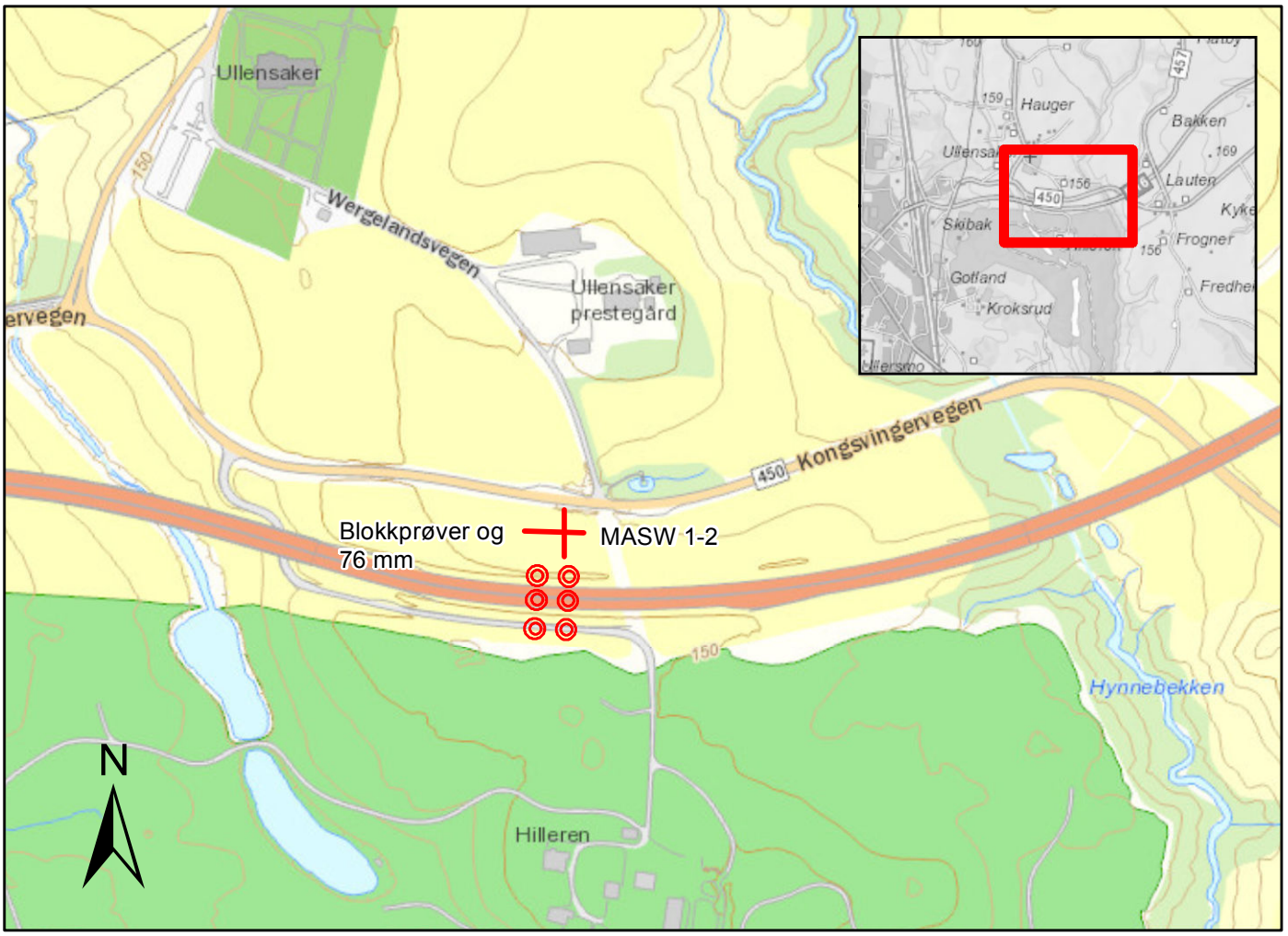
	Kontrollert	Godkjent



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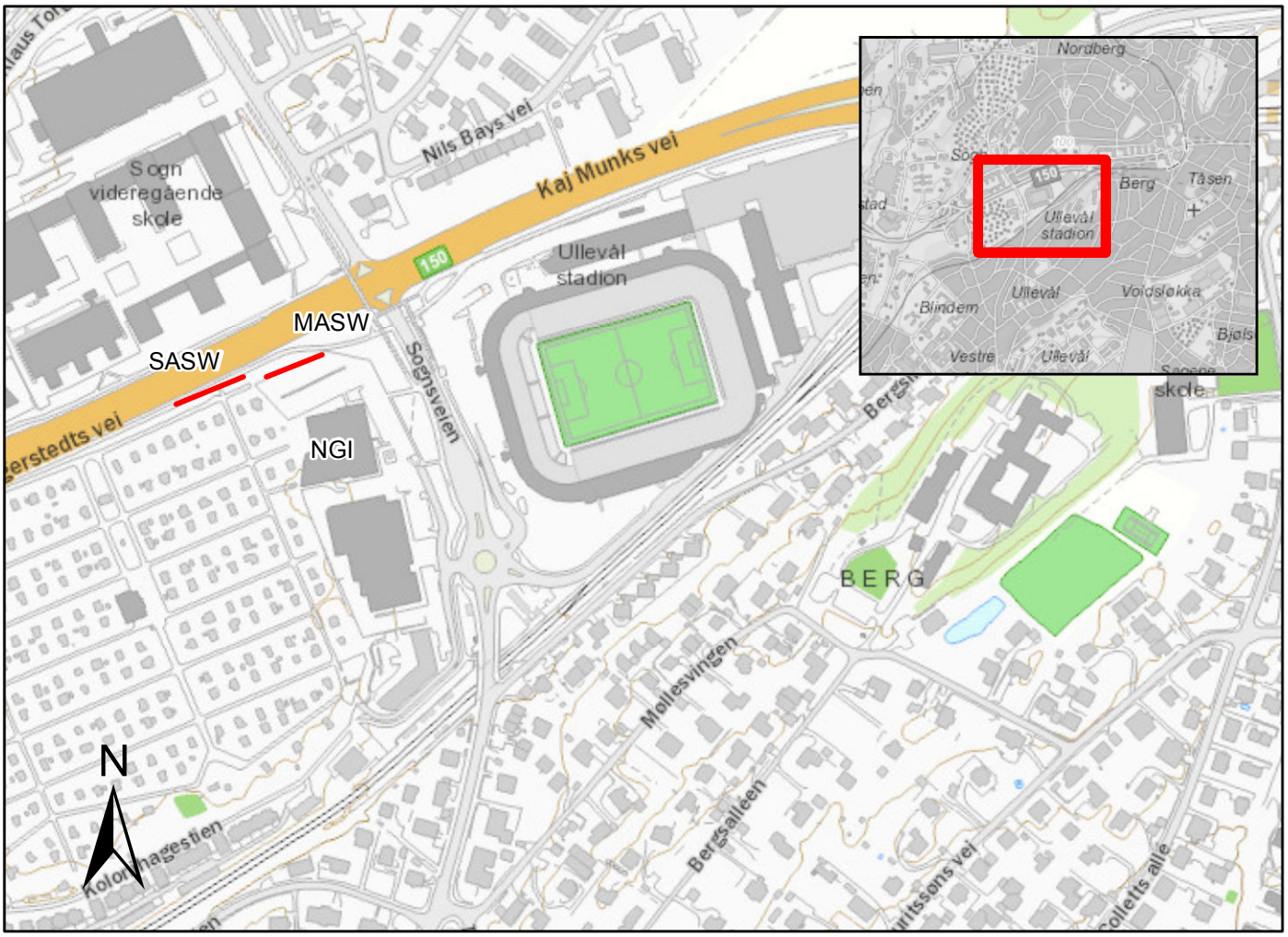
Målestokk (A4): 1:5 000 Kartprojeksjon: UTM_Zone_33N

RVII - Hilleren		
MASW og blokkprøvelokalitet	Prosjektnr. **20140622**	Kart nr. **6**
	Utført **JSL**	Dato **2015-02-24**
	Kontrollert	Godkjent
	NGI	



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Målestokk (A4): 1:5 000 Kartprojeksjon: UTM_Zone_33N

NGI-car park		
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	Kontrollert	Godkjent
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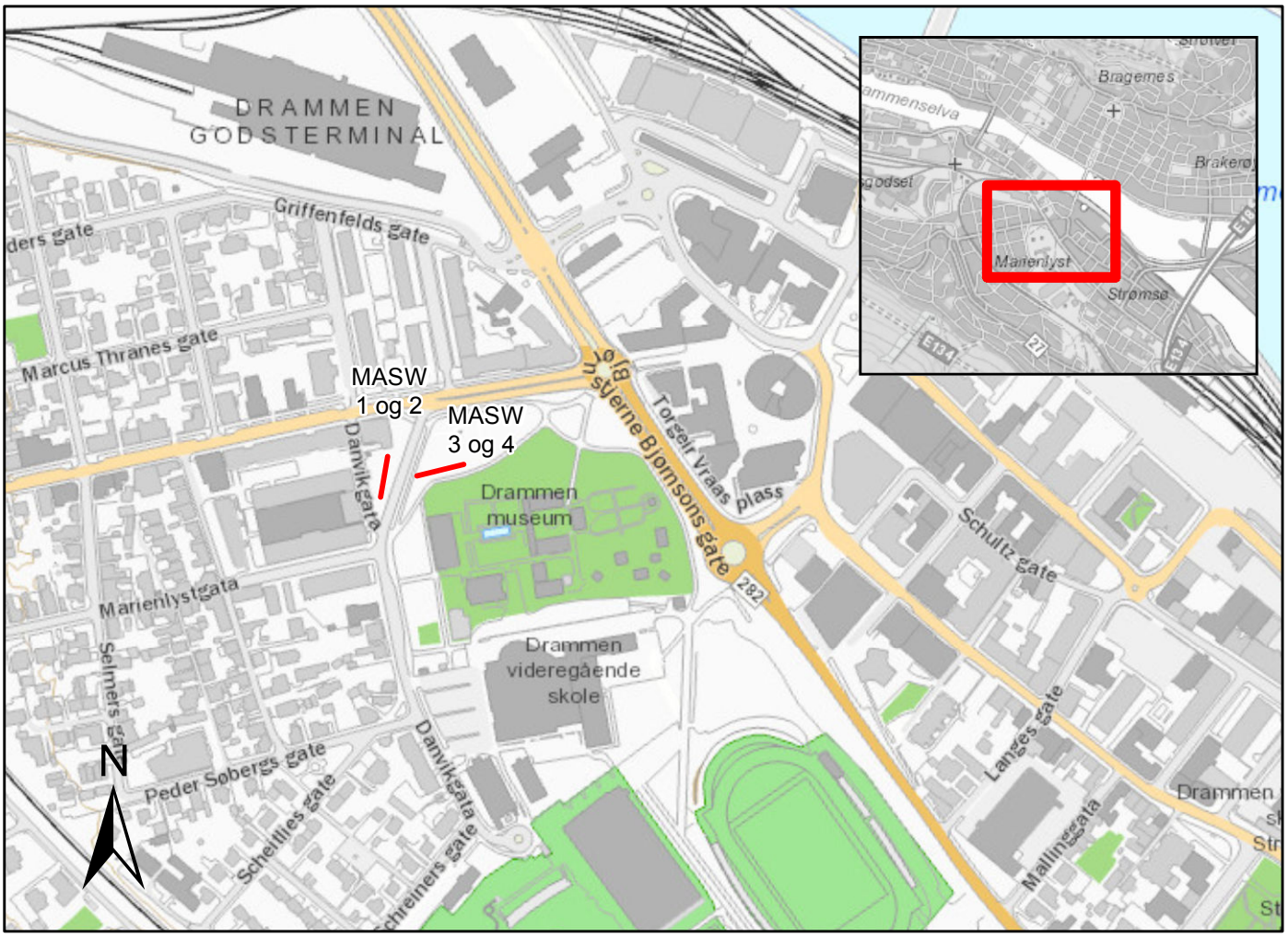
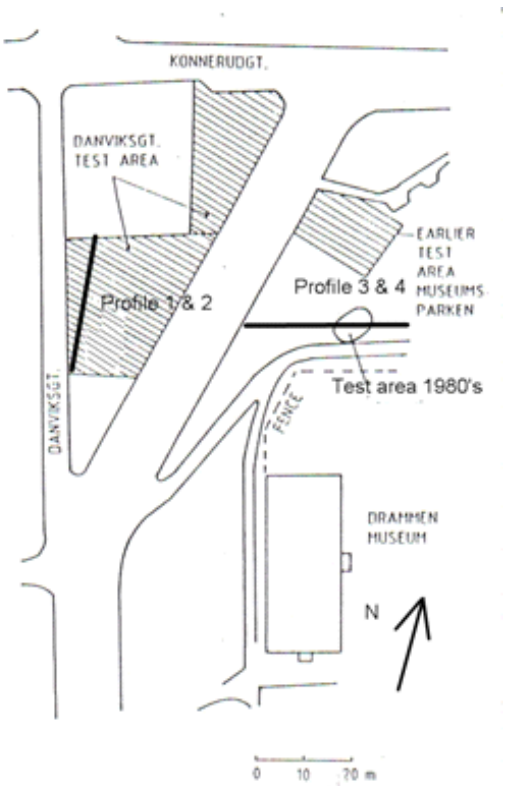
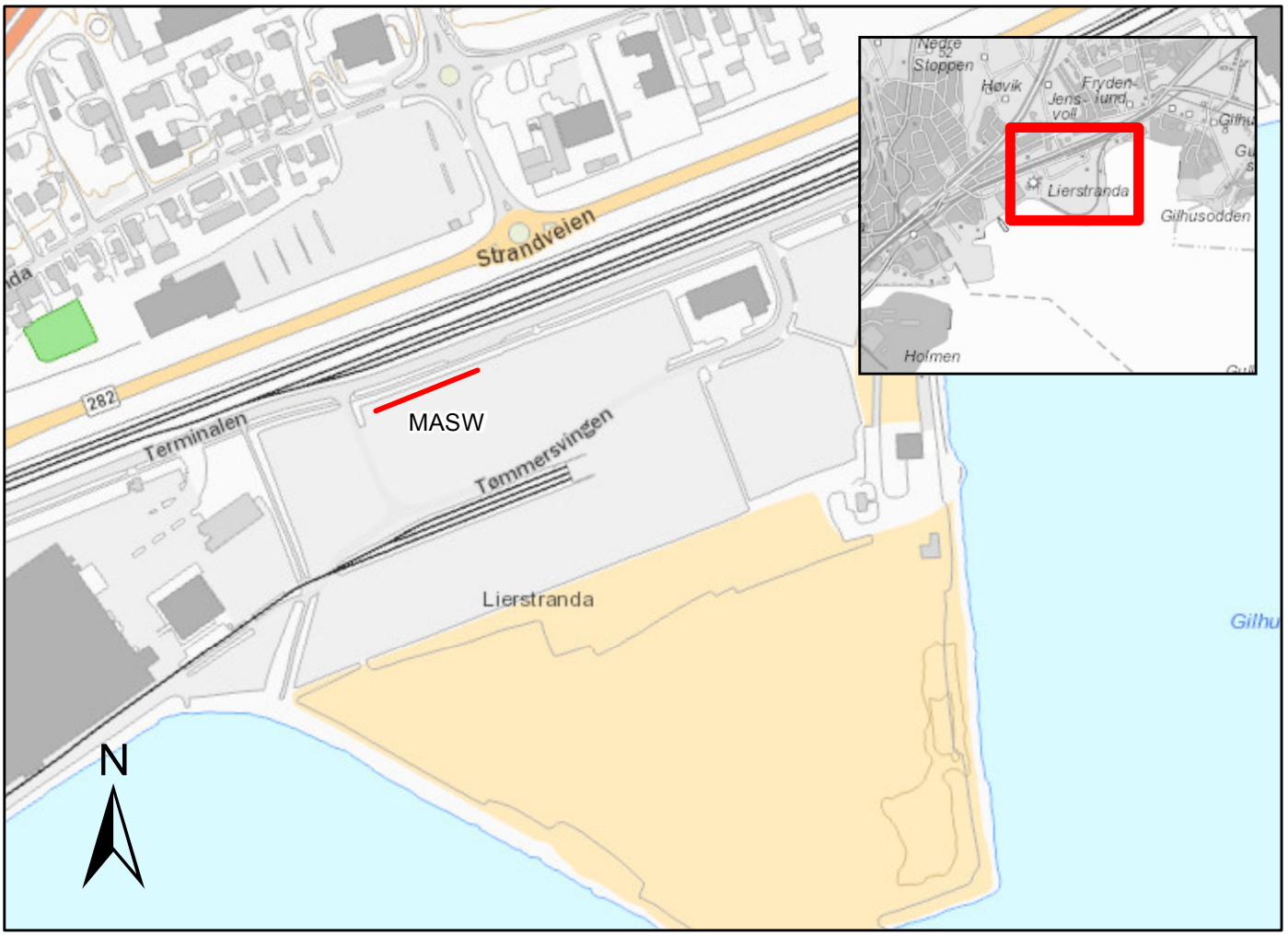




Målestokk (A4): 1:5 000 Kartprojeksjon: UTM_Zone_33N

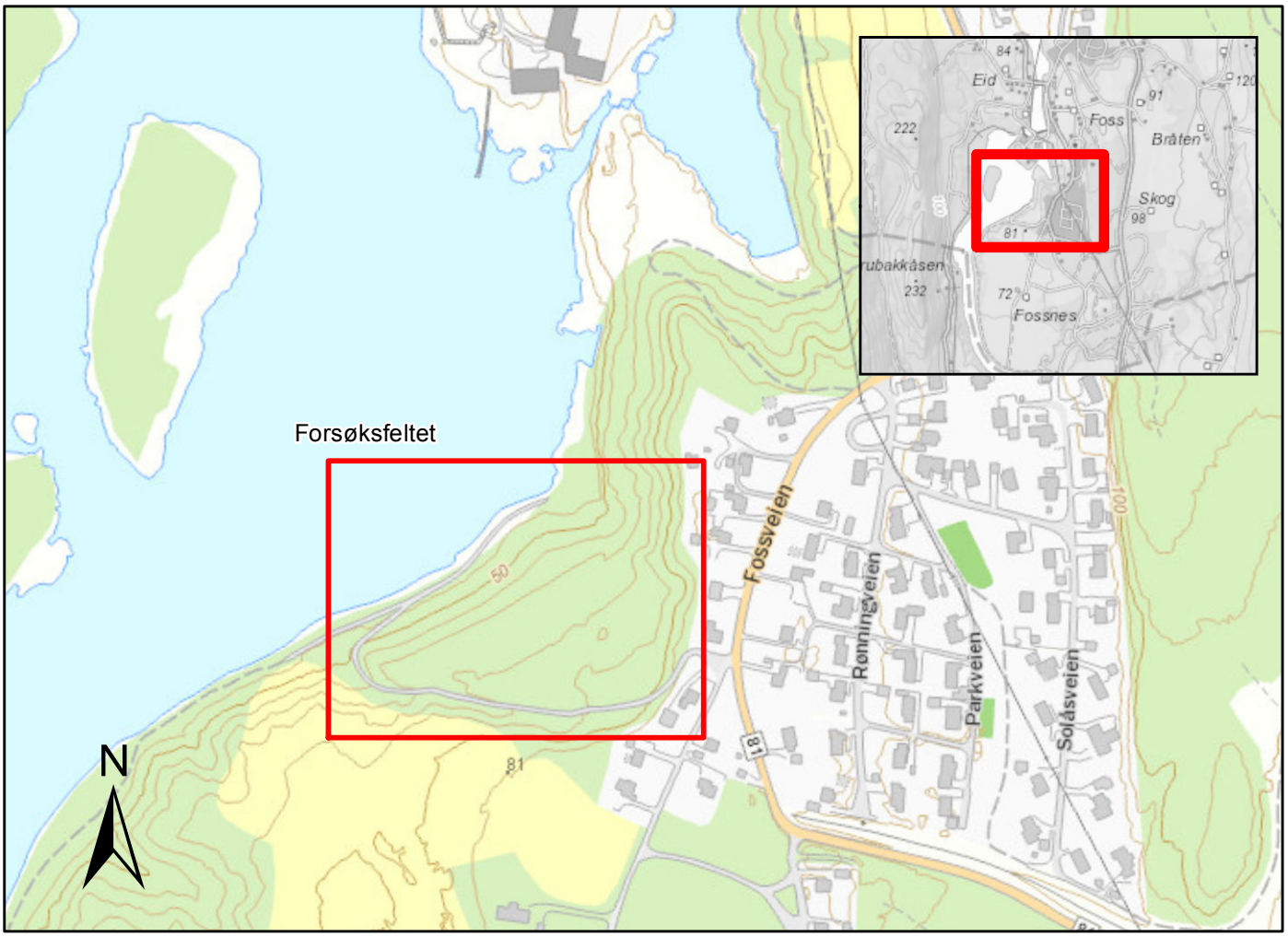
Danviksgata / Museumpark Drammen		
MASW	Prosjektnr. 20140622	Kart nr. 8
	Utført JSL	Dato 2015-02-24
	Kontrollert	Godkjent
	NGI	

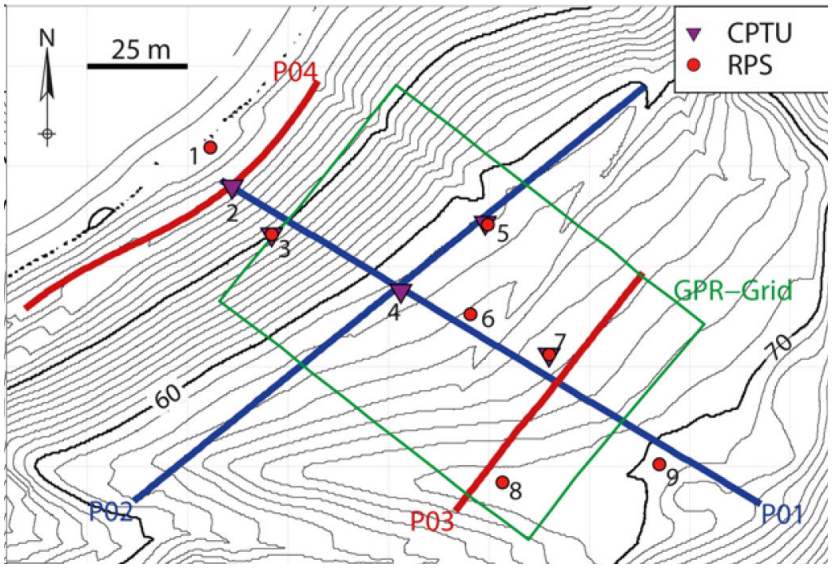


0 150 300 m

Målestokk (A4): 1:5 000 Kartprojeksjon: UTM_Zone_33N

Lierstranda - Drammen		
MASW	Prosjektnr. 20140622	Kart nr. 9
	Utført JSL	Dato 2015-02-24
	Kontrollert	Godkjent
	NGI	



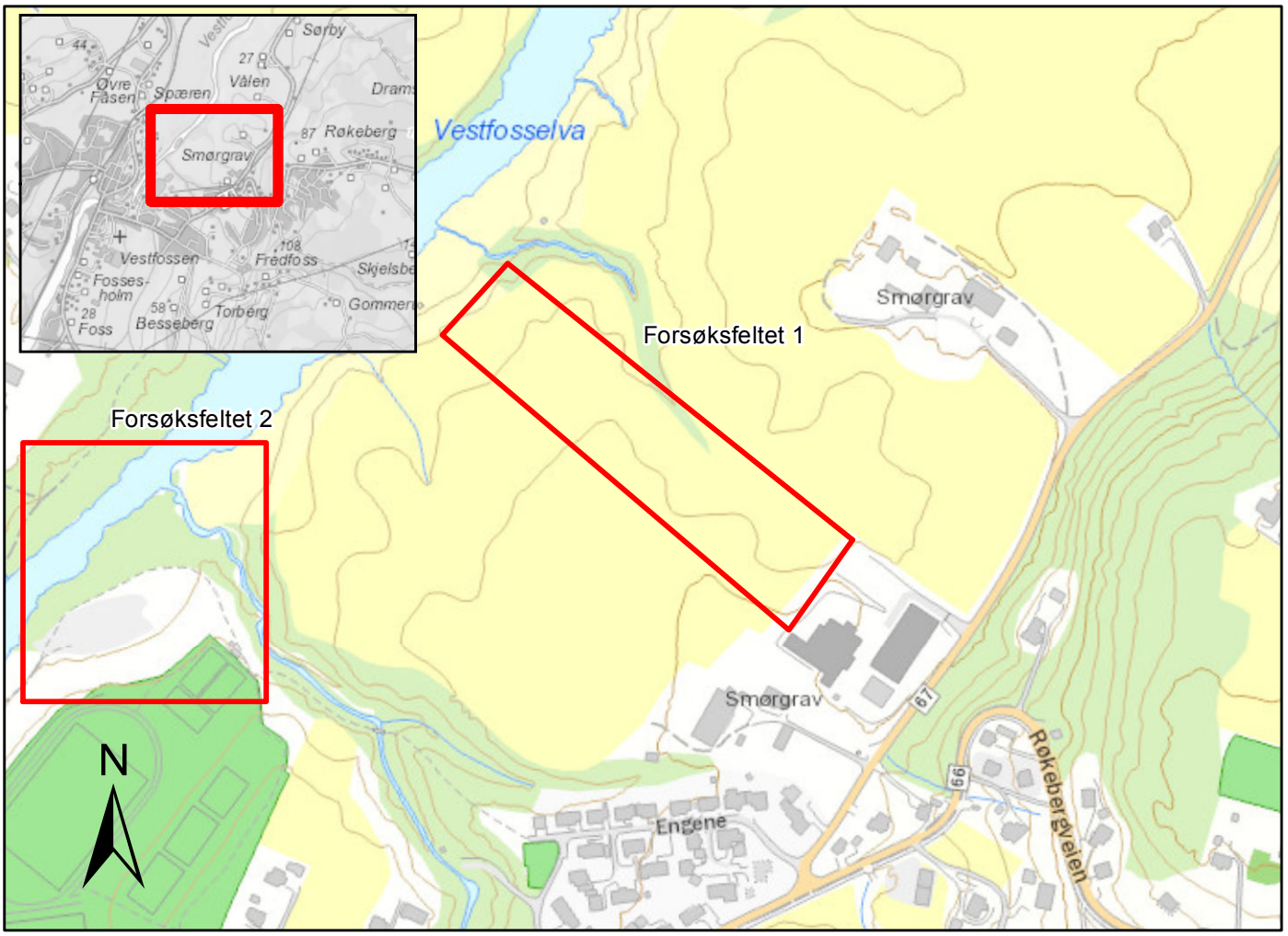


0 150 300 m

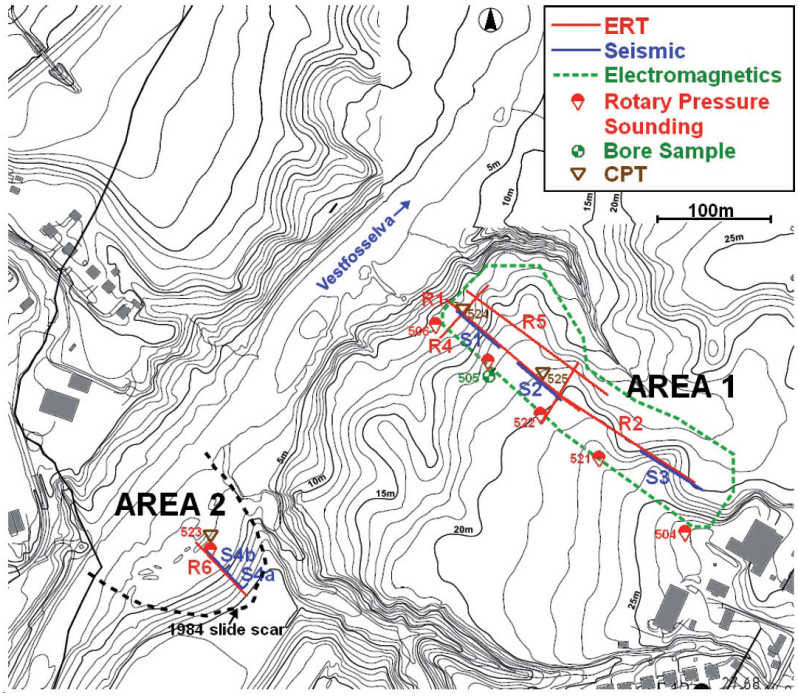
Målestokk (A4): 1:5 000 Kartprojeksjon: UTM_Zone_33N

Hvittingfoss		
Forsøksfelt	Prosjektnr. 20140622	Kart nr. 10
	Utført JSL	Dato 2015-02-24
	Kontrollert	Godkjent
	NGI	

Red and blue lines: MASW

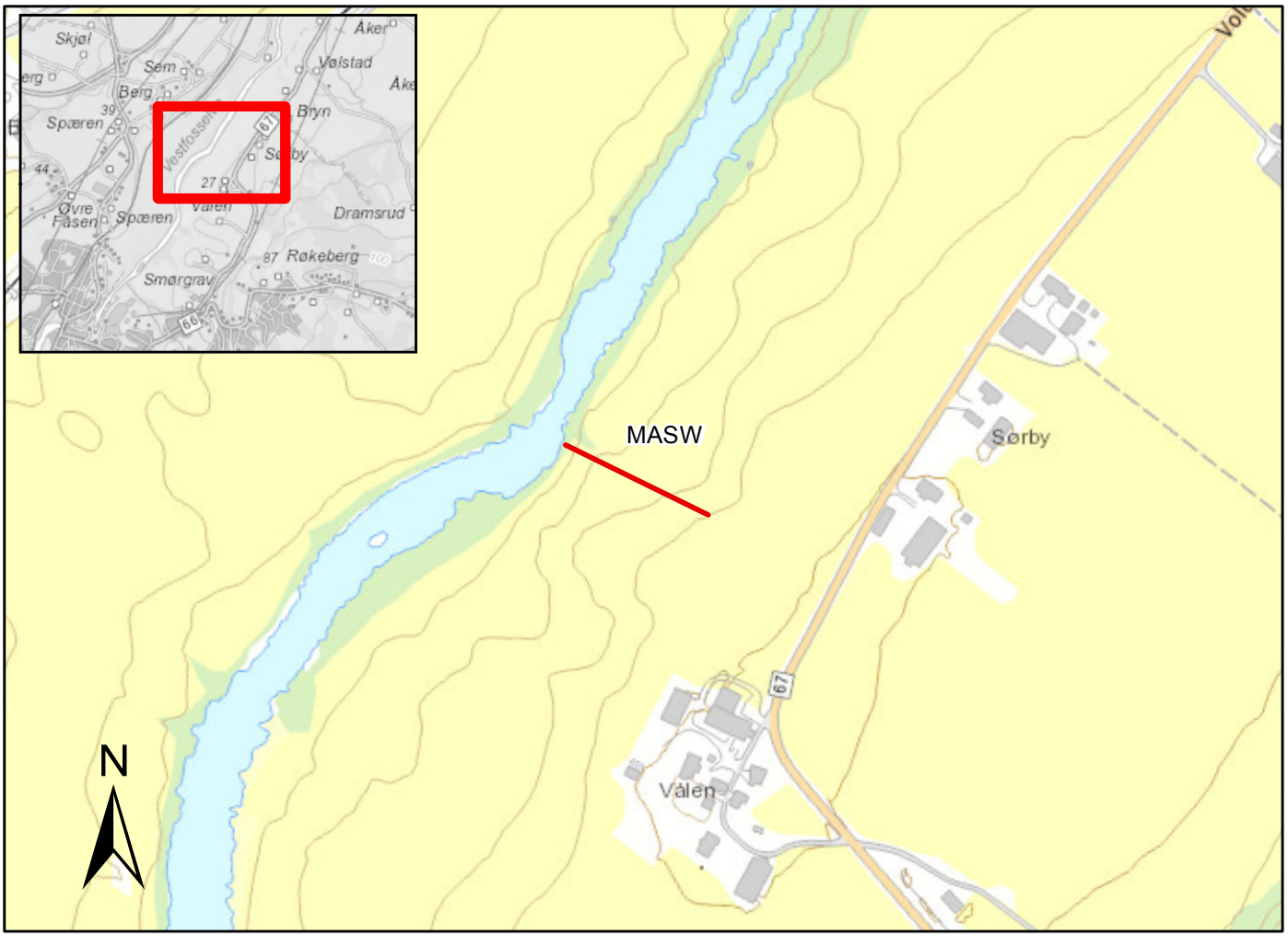


0 150 300 m

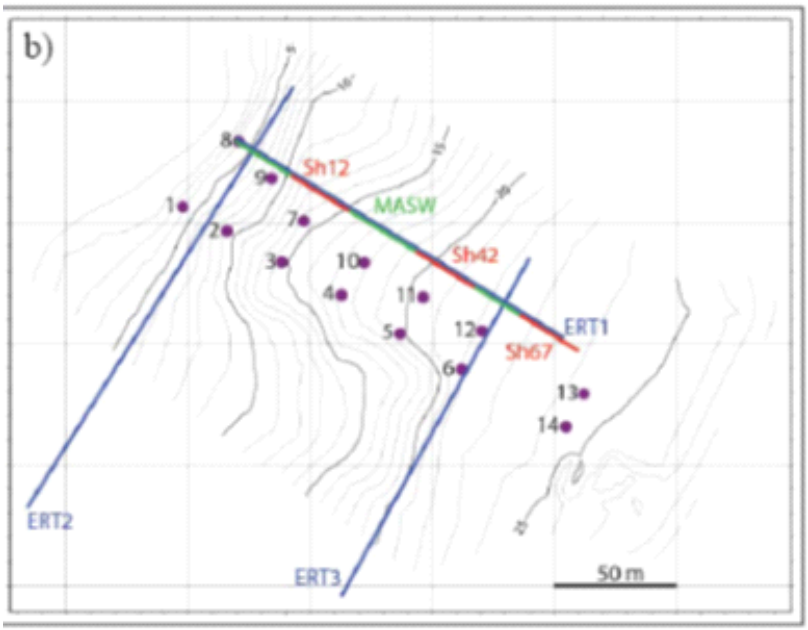


Målestokk (A4): 1:5 000 Kartprojeksjon: UTM_Zone_33N

Smørgrav		
Forsøksfelt 1 - 2	Prosjektnr. 20140622	Kart nr. 11
	Utført JSL	Dato 2015-02-24
	Kontrollert	Godkjent

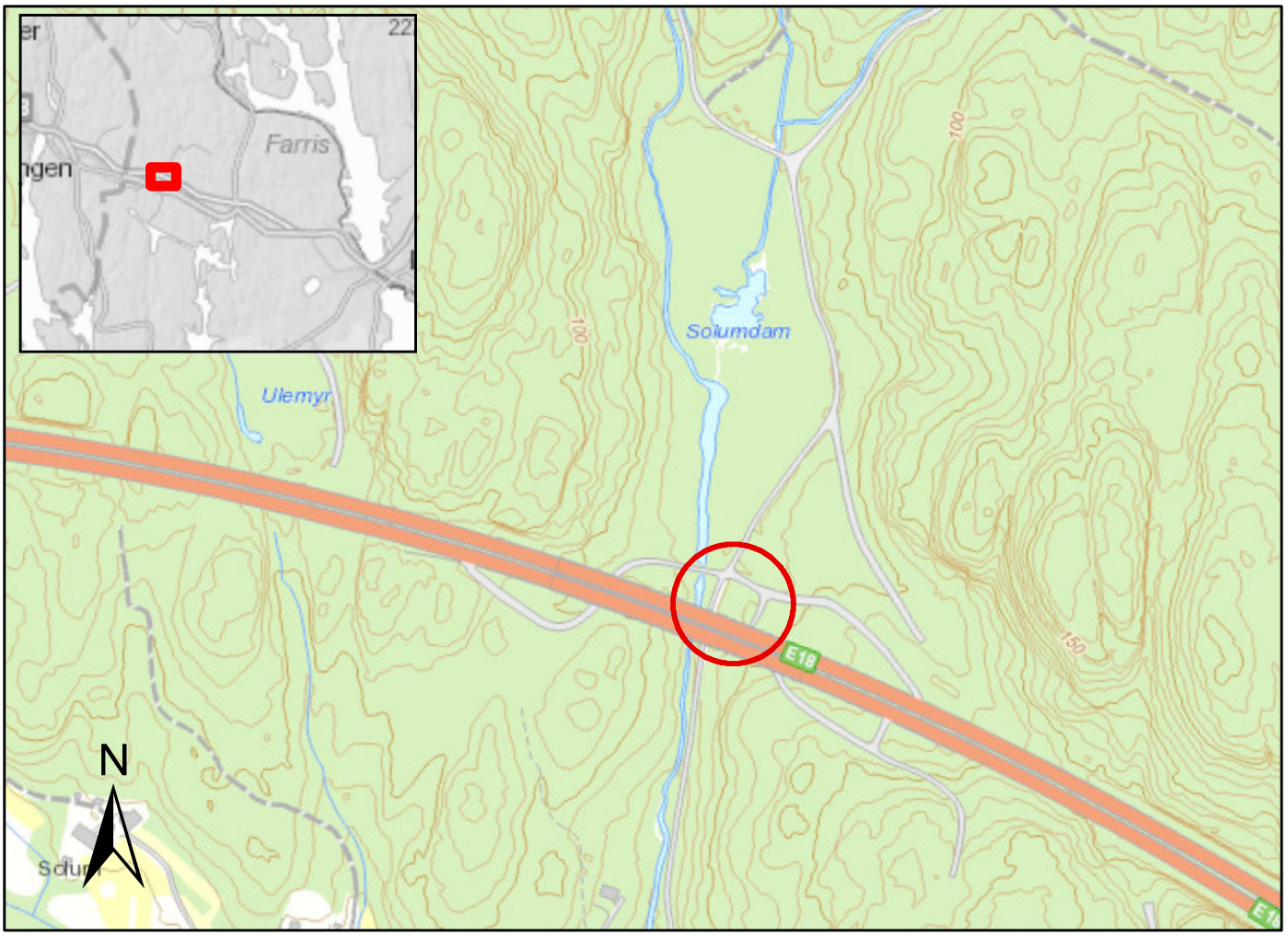


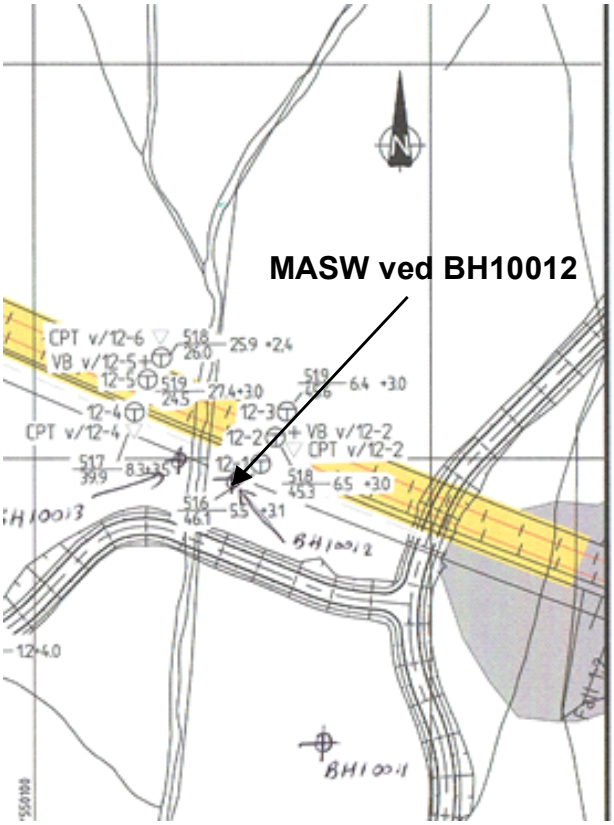
0 150 300 m



Målestokk (A4): 1:5 000 Kartprojeksjon: UTM_Zone_33N

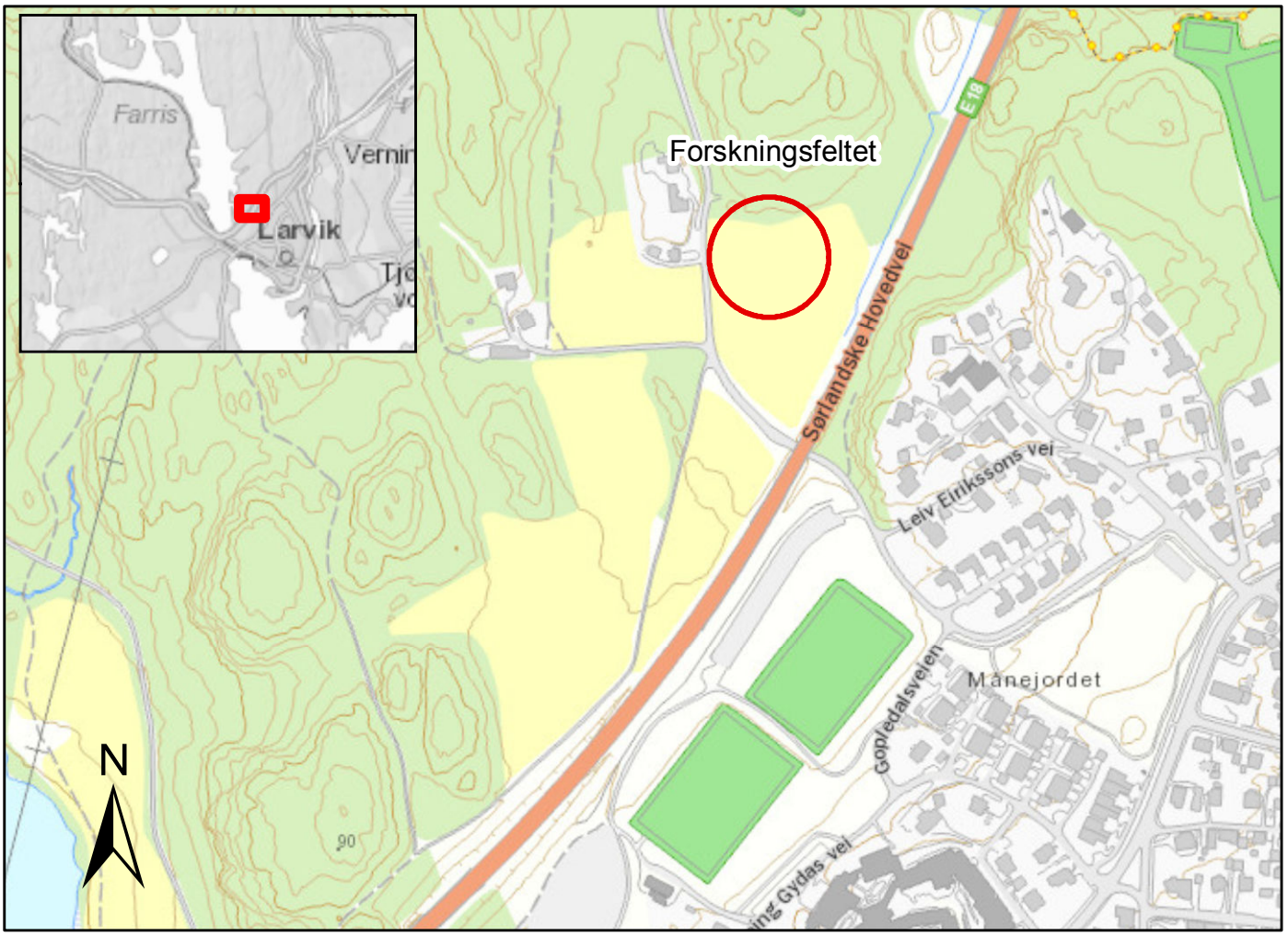
Vålen		
Forsøksfelt 1 - 2	Prosjektnr. 20140622	Kart nr. 12
	Utført JSL	Dato 2015-02-24
	Kontrollert	Godkjent
	NGI	





Målestokk (A4): 1:5 000 Kartprojeksjon: UTM_Zone_33N

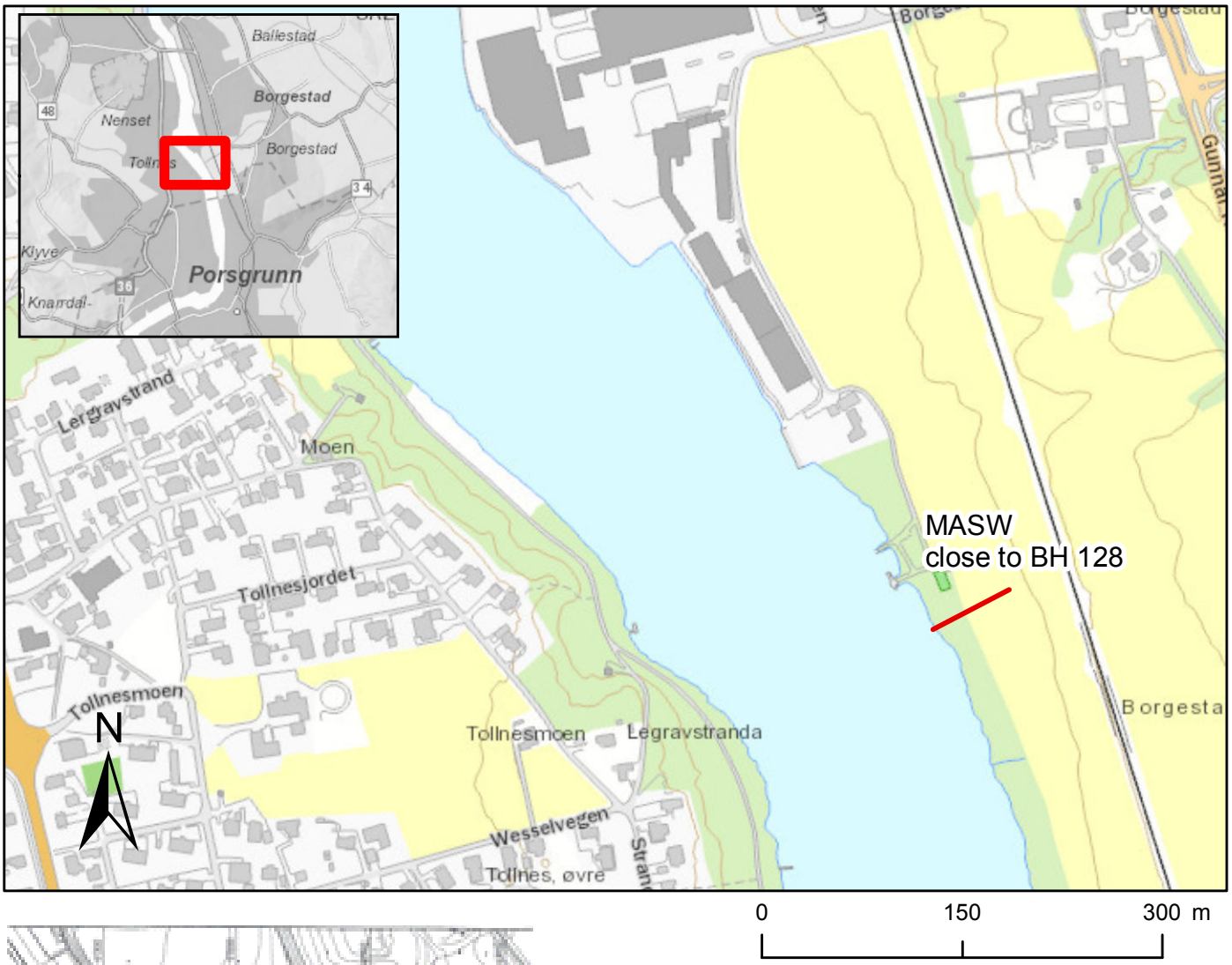
Farriseidet		
MASW og borpunkter	Prosjektnr. 20140622	Kart nr. 13
	Utført JSL	Dato 2015-02-24
	Kontrollert	Godkjent
	NGI	

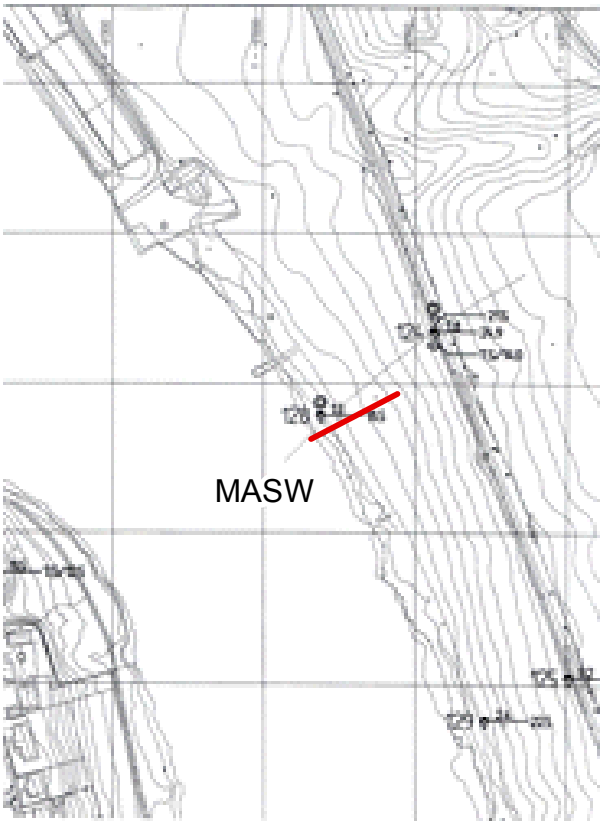


0 150 300 m

Målestokk (A4): 1:5 000 Kartprojeksjon: UTM_Zone_33N

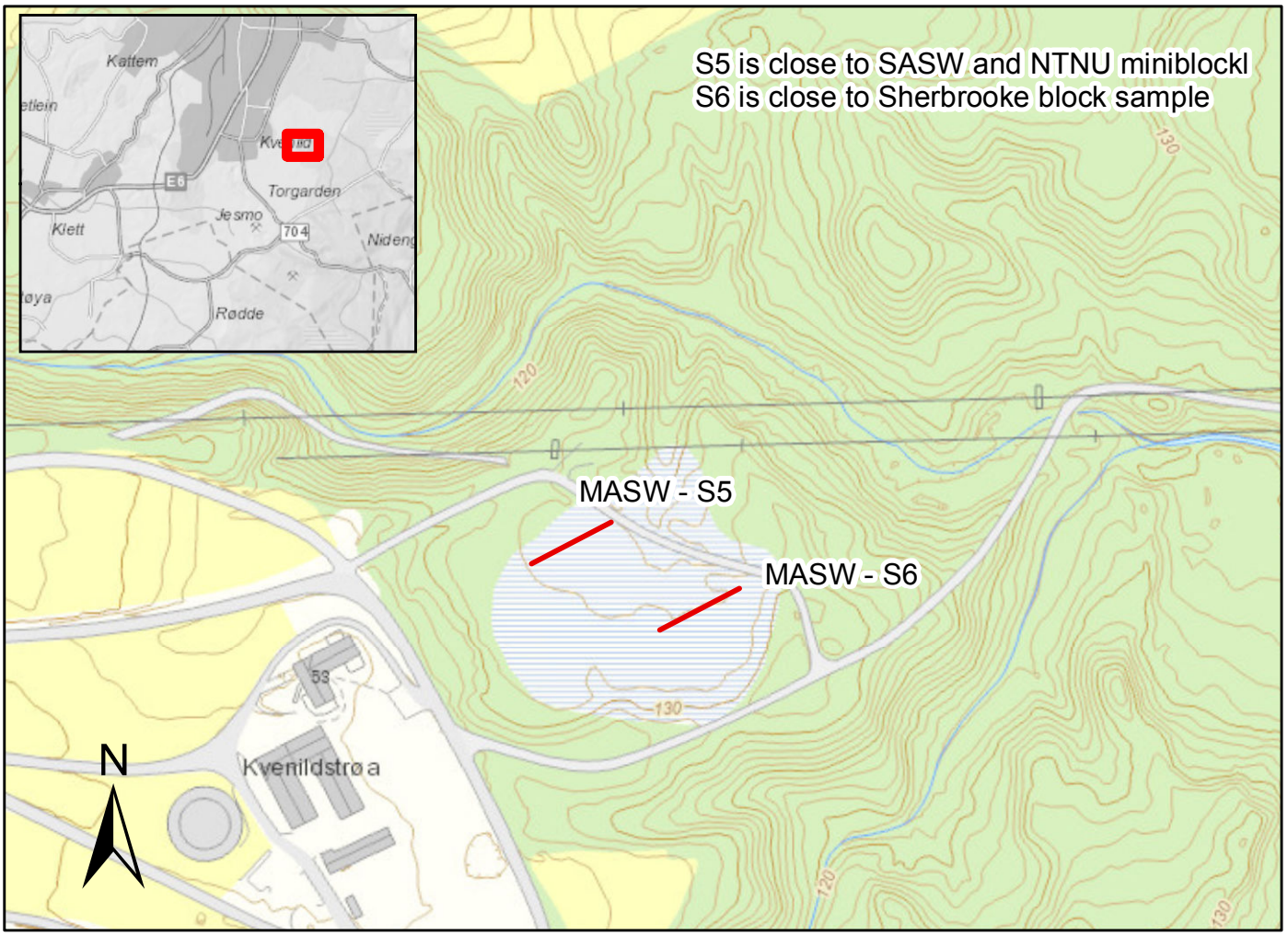
Månejordet - Farriseidet		
Forskningsfelt	Prosjektnr. 20140622	Kart nr. 14
	Utført JSL	Dato 2015-02-24
	Kontrollert	Godkjent
	NGI	



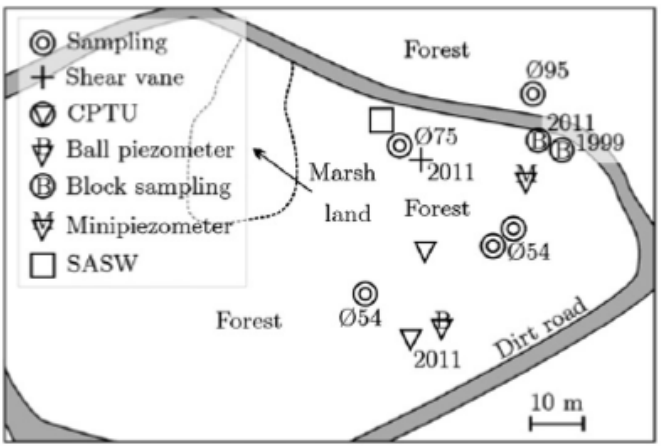


Målestokk (A4): 1:5 000 Kartprojeksjon: UTM_Zone_33N

Skienselven		
MASW og borepunkter	Prosjektnr. 20140622	Kart nr. 15
	Utført JSL	Dato 2015-02-24
	Kontrollert	Godkjent
	NGI	

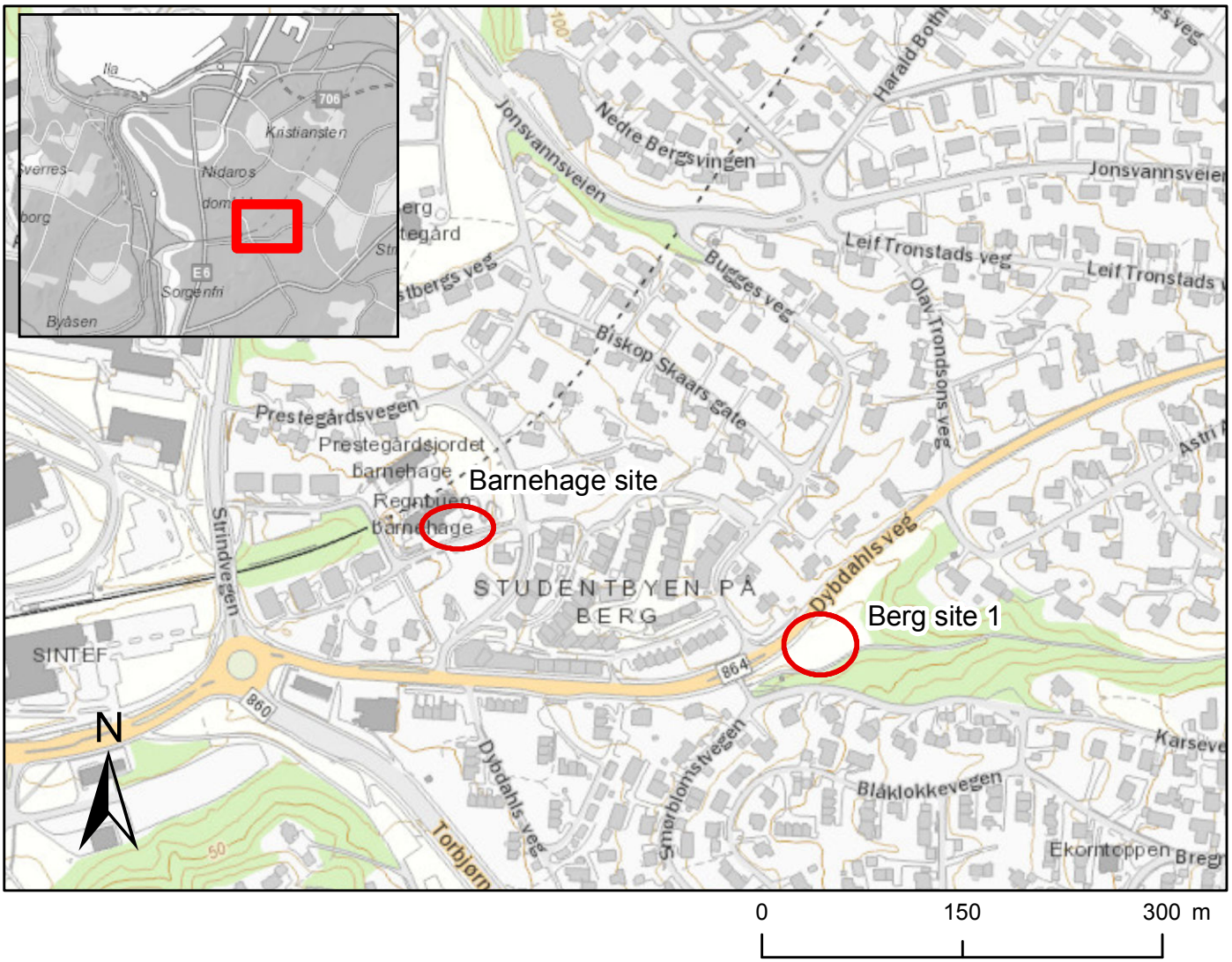


0 75 150 m



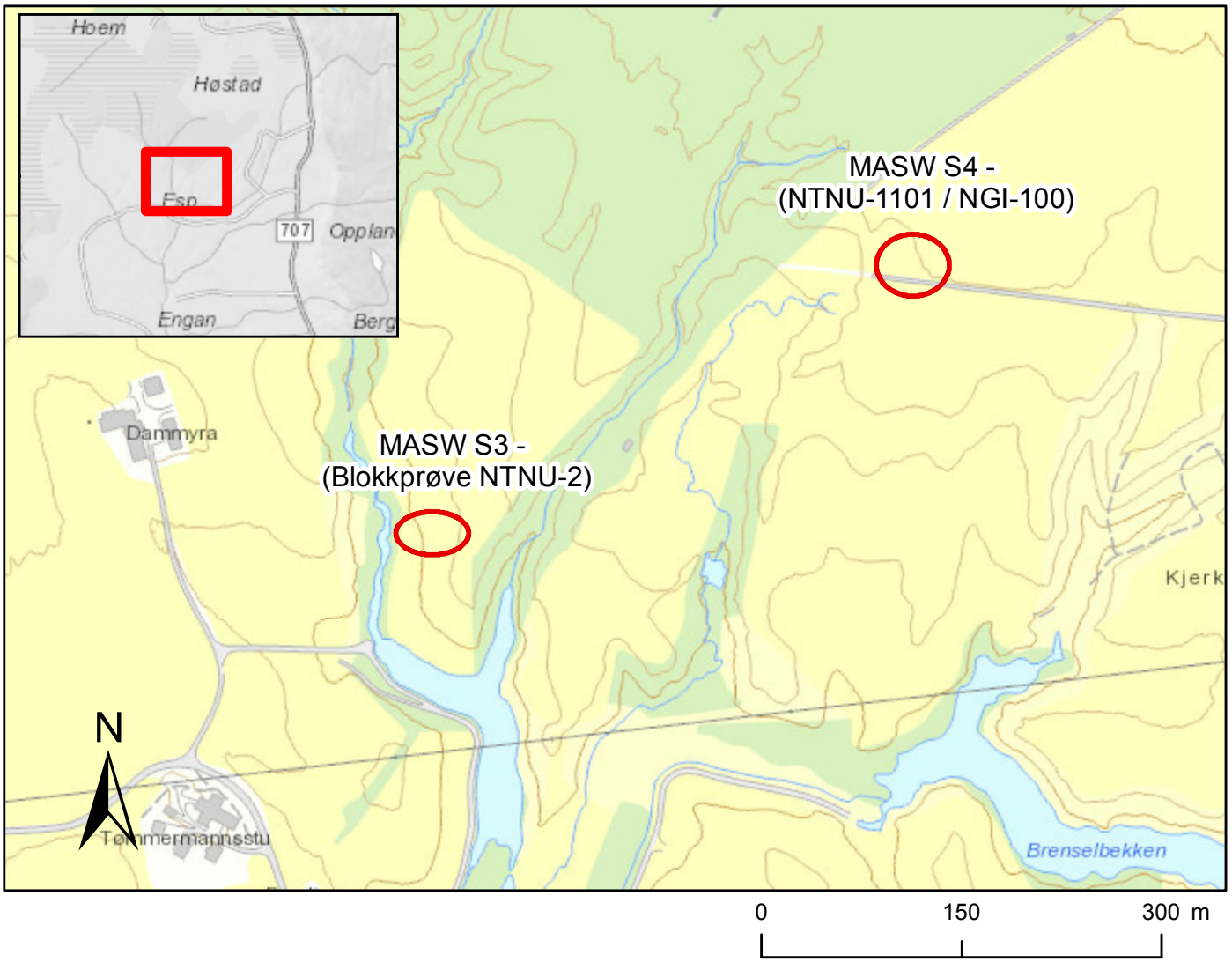
Målestokk (A4): 1:2 500 Kartprojeksjon: UTM_Zone_33N

Tiller		
MASW, SCPTU og borepunkter	Prosjektnr. **20140622**	Kart nr. **16**
	Utført **JSL**	Dato **2015-02-24**
	Kontrollert	Godkjent
	NGI	



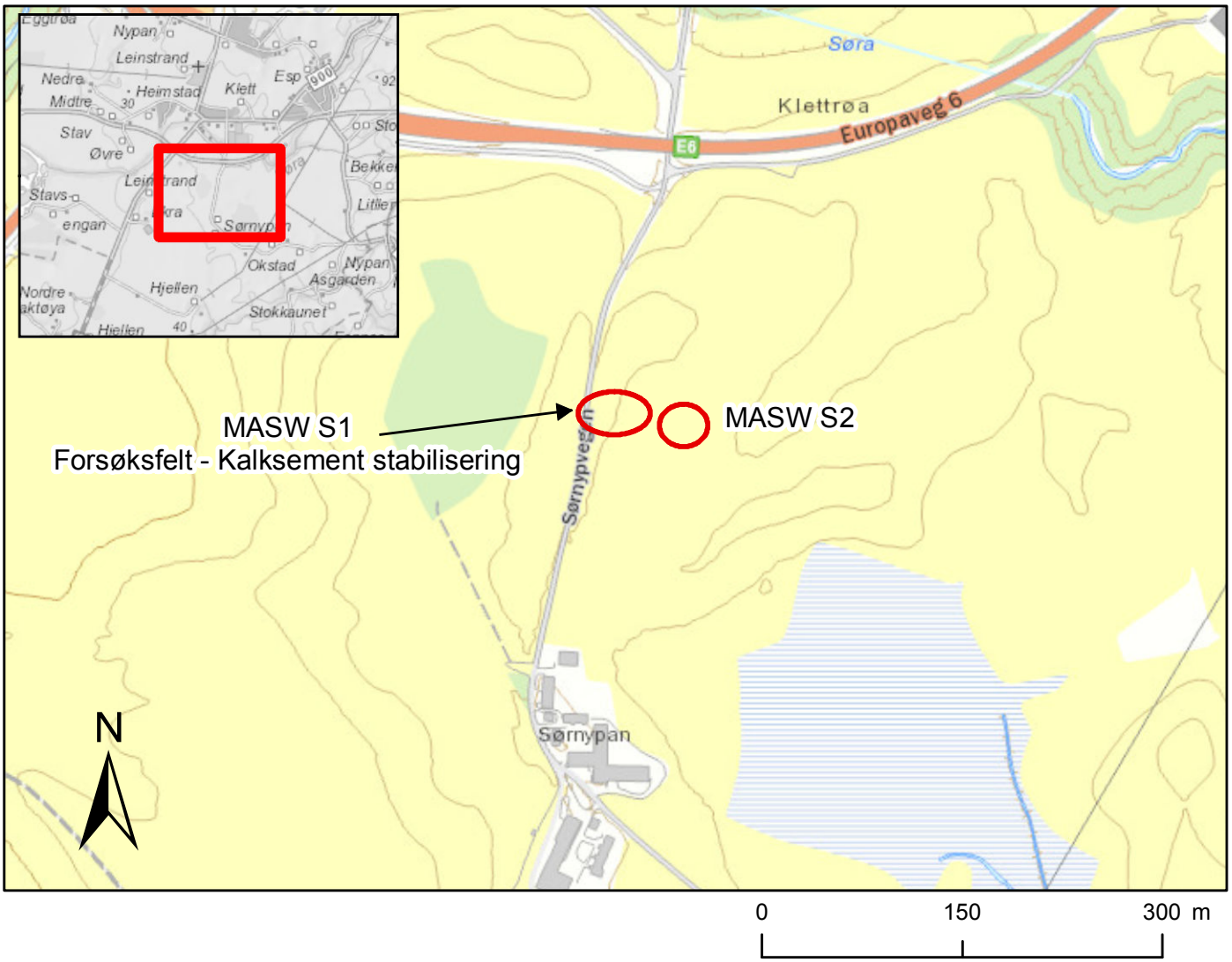
Målestokk (A4): 1:5 000 Kartprojeksjon: UTM_Zone_33N

Berg, Trondheim		
MASW/crosshole tests	Prosjektnr. **20140622**	Kart nr. **17**
Data from the barnehage site and site 1 are compared	Utført **JSL**	Dato **2015-02-24**
	Kontrollert	Godkjent
	NGI	



Målestokk (A4): 1:5 000 Kartprojeksjon: UTM_Zone_33N

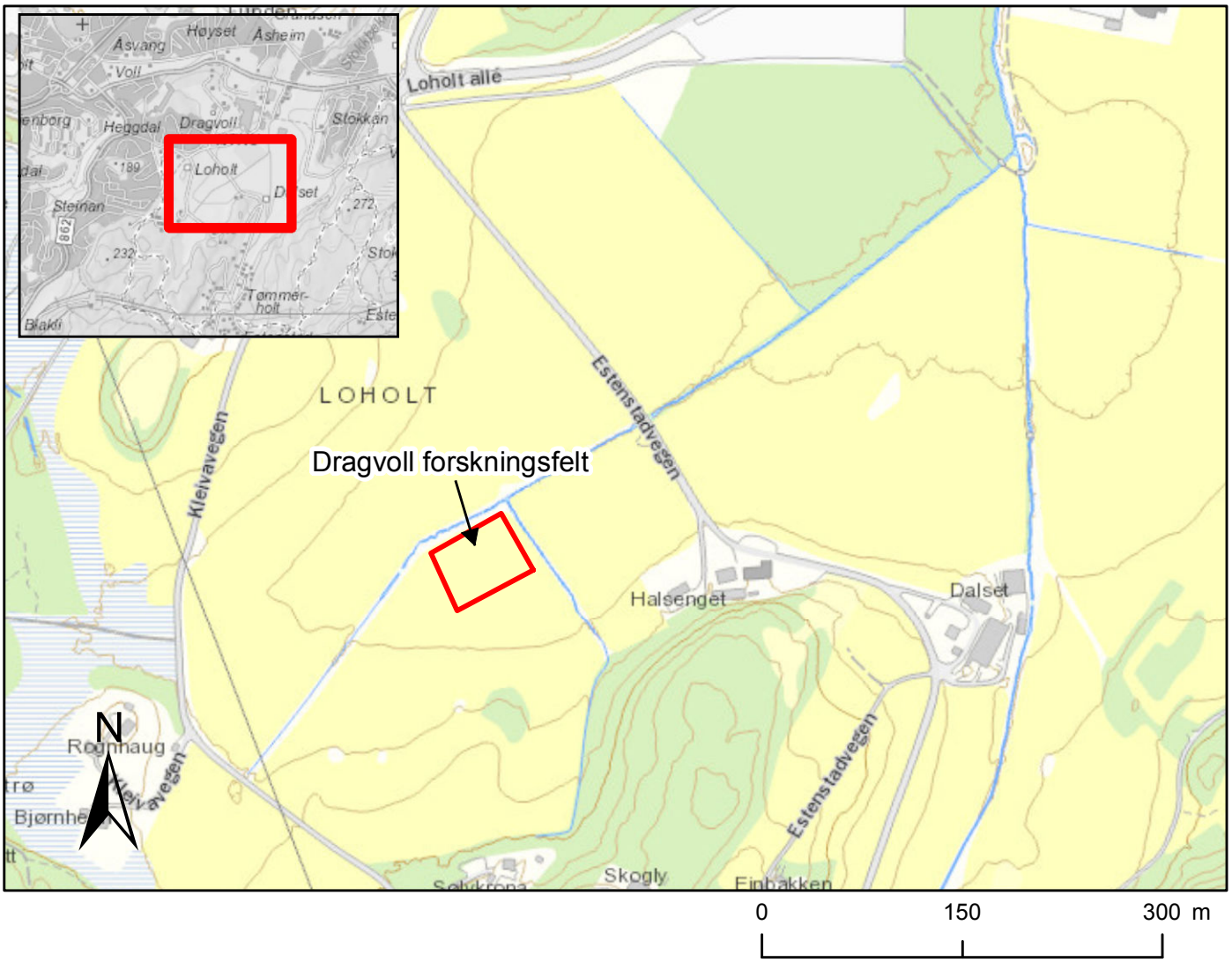
Esp, Byneset		
MASW og prøvetaking	Prosjektnr. 20140622	Kart nr. 18
	Utført JSL	Dato 2015-02-24
	Kontrollert	Godkjent
	NGI	



0 150 300 m

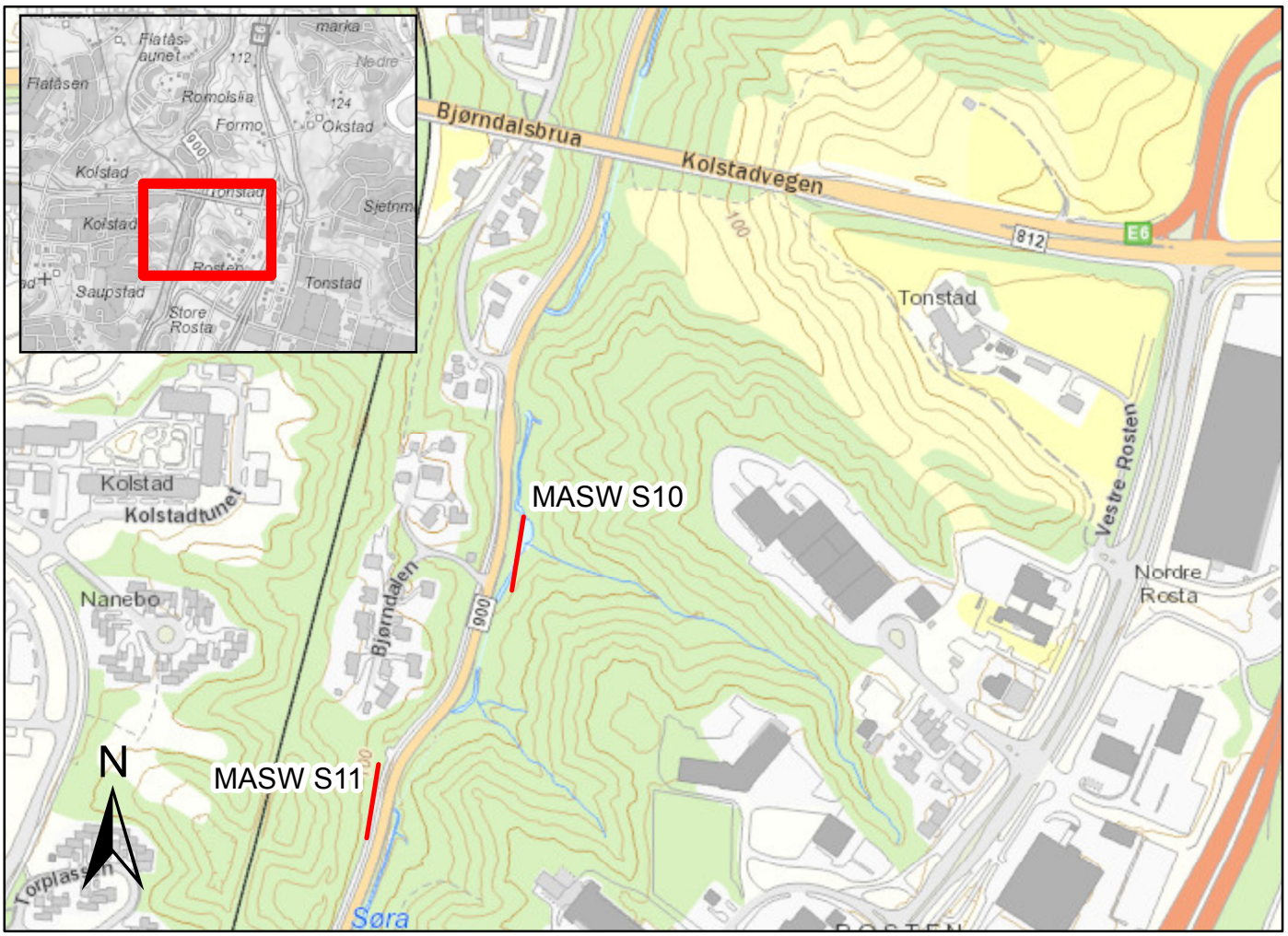
Målestokk (A4): 1:5 000 Kartprojeksjon: UTM_Zone_33N

Klett, Trondheim		
Forsøksfelt med MASW	Prosjektnr. 20140622	Kart nr. 19
	Utført JSL	Dato 2015-02-24
	Kontrollert	Godkjent
	NGI	



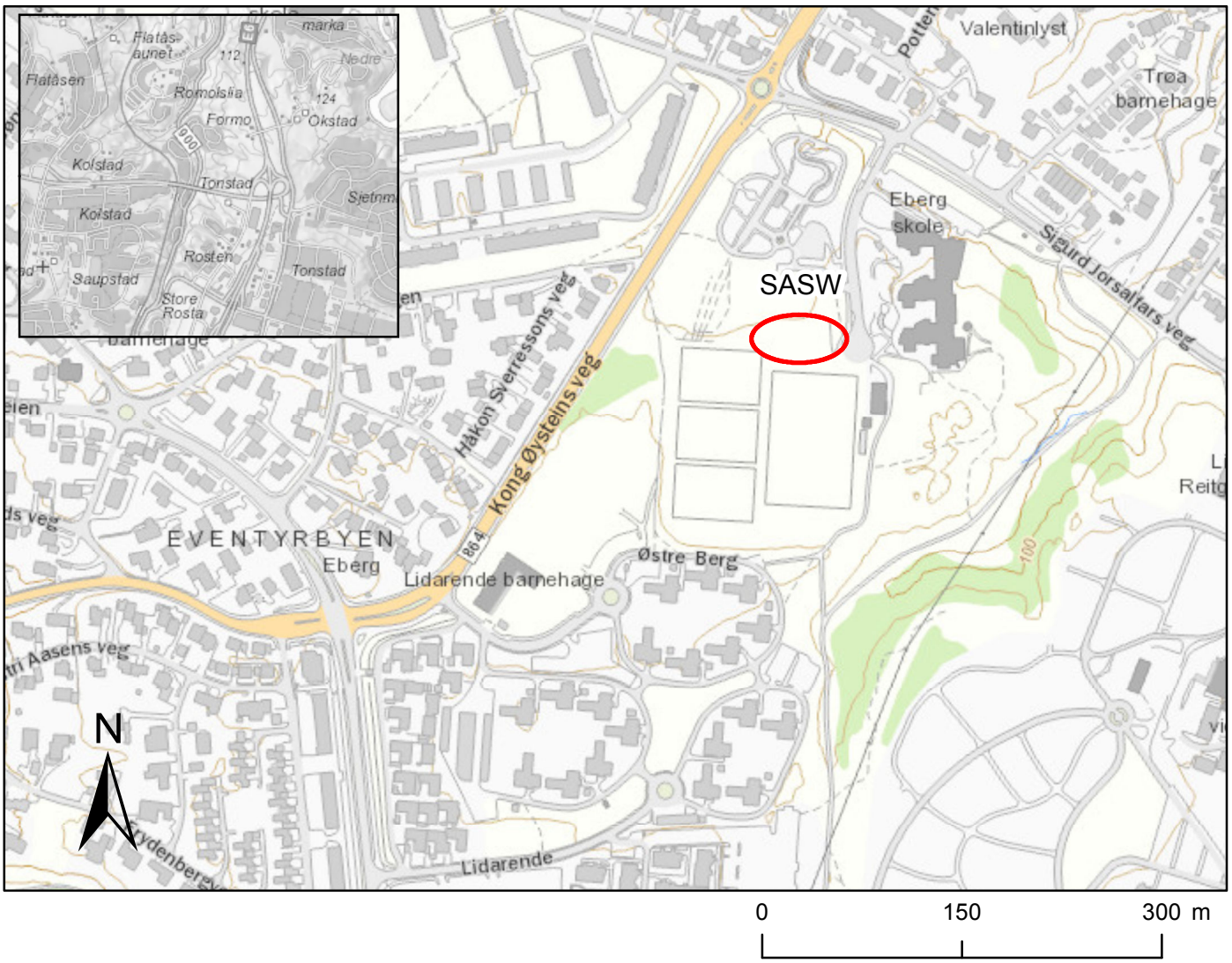
Målestokk (A4): 1:5 000 Kartprojeksjon: UTM_Zone_33N

Dragvoll, Trondheim		
Forsøksfelt med MASW	Prosjektnr. 20140622	Kart nr. 20
	Utført JSL	Dato 2015-02-24
	Kontrollert	Godkjent
	NGI	



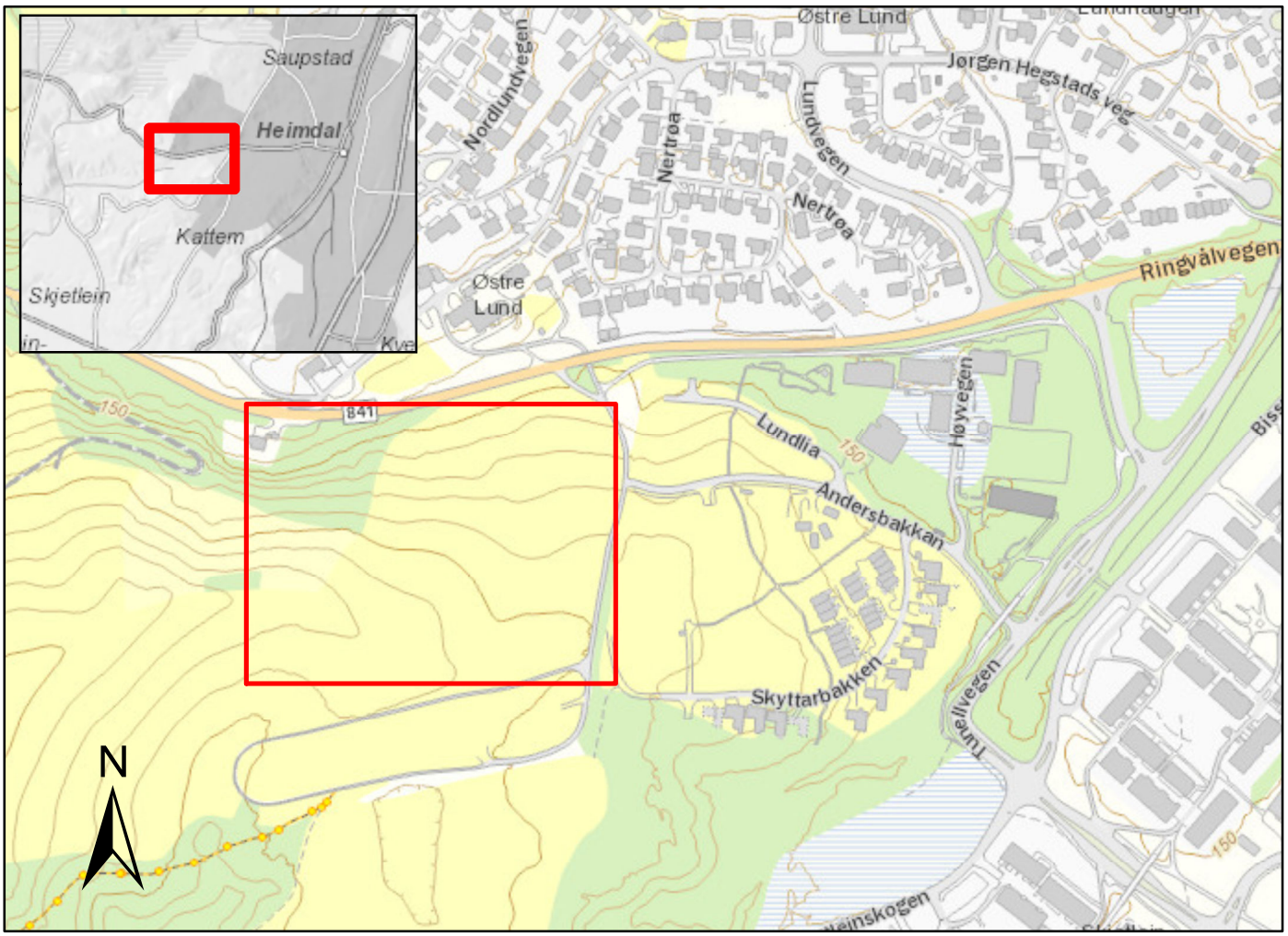
Målestokk (A4): 1:5 000 Kartprojeksjon: UTM_Zone_33N

Rosten-Saupstad, Trondheim		
Forsøksfelt med MASW	Prosjektnr. **20140622**	Kart nr. **21-22**
	Utført **JSL**	Dato **2015-02-24**
	Kontrollert	Godkjent
	NGI	



Målestokk (A4): 1:5 000 Kartprojeksjon: UTM_Zone_33N

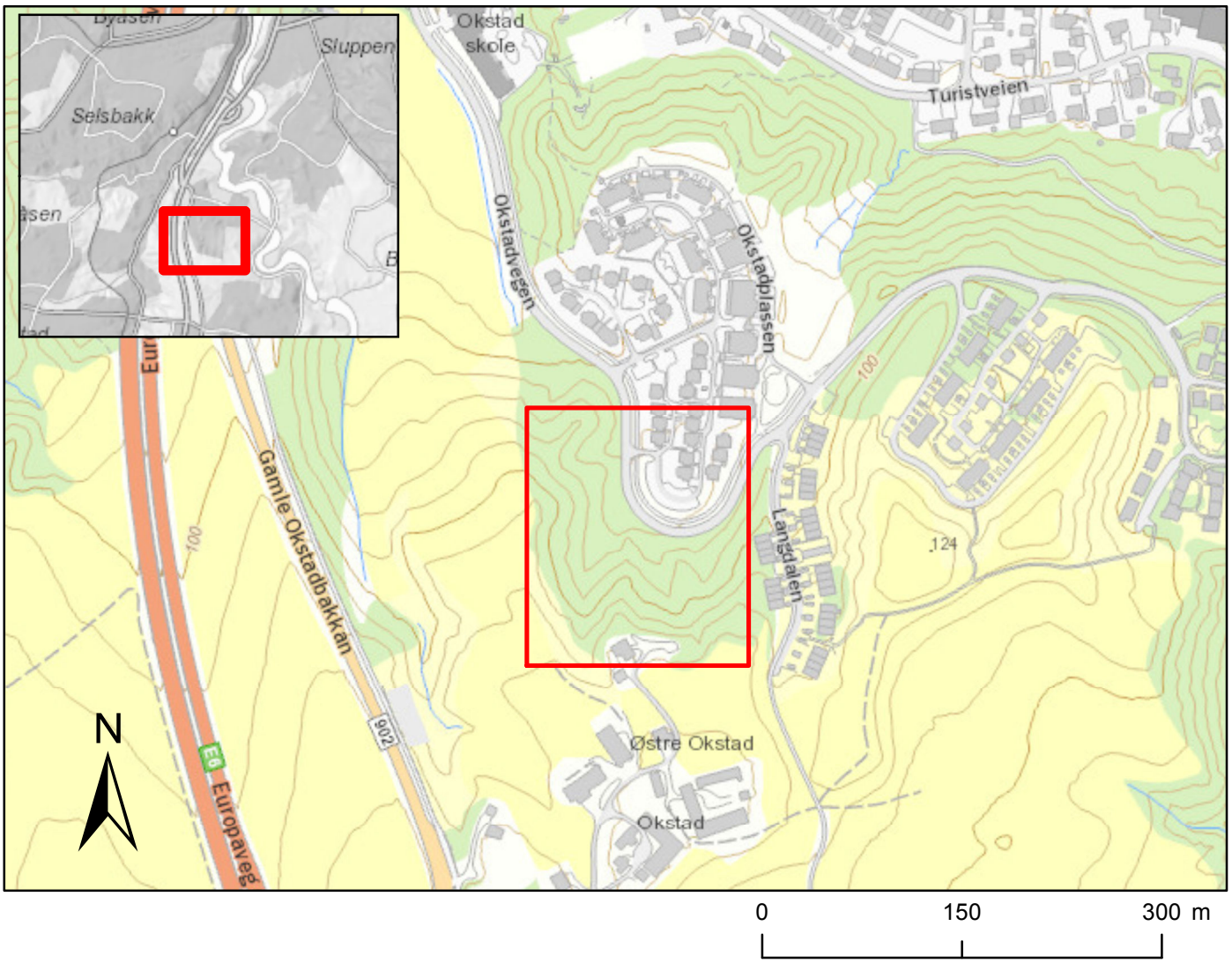
Eberg, Trondheim		
Forsøksfelt - SASW	Prosjektnr. 20140622	Kart nr. 23
	Utført JSL	Dato 2015-02-24
	Kontrollert	Godkjent
	NGI	



0 150 300 m

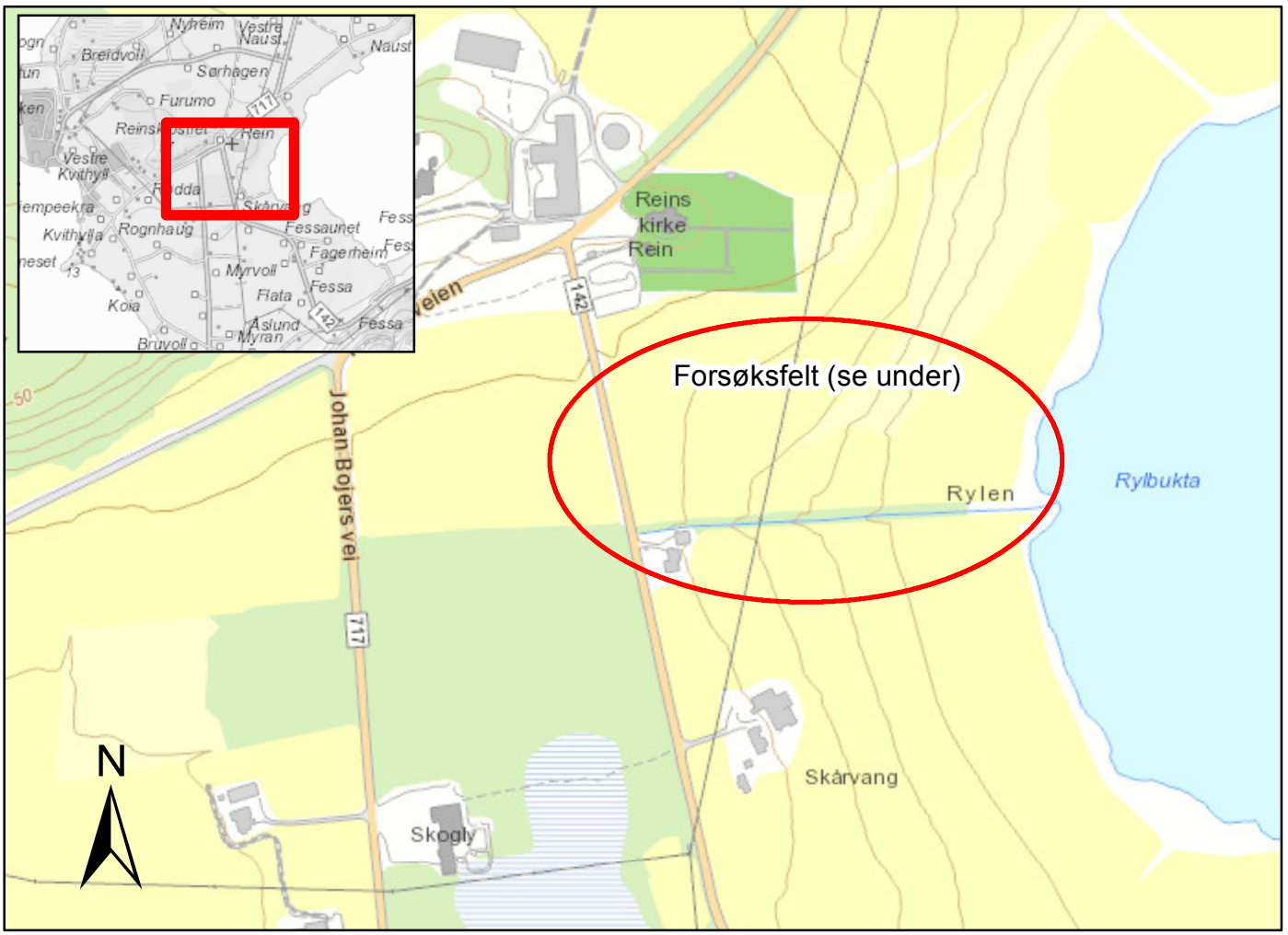
Målestokk (A4): 1:5 000 Kartprojeksjon: UTM_Zone_33N

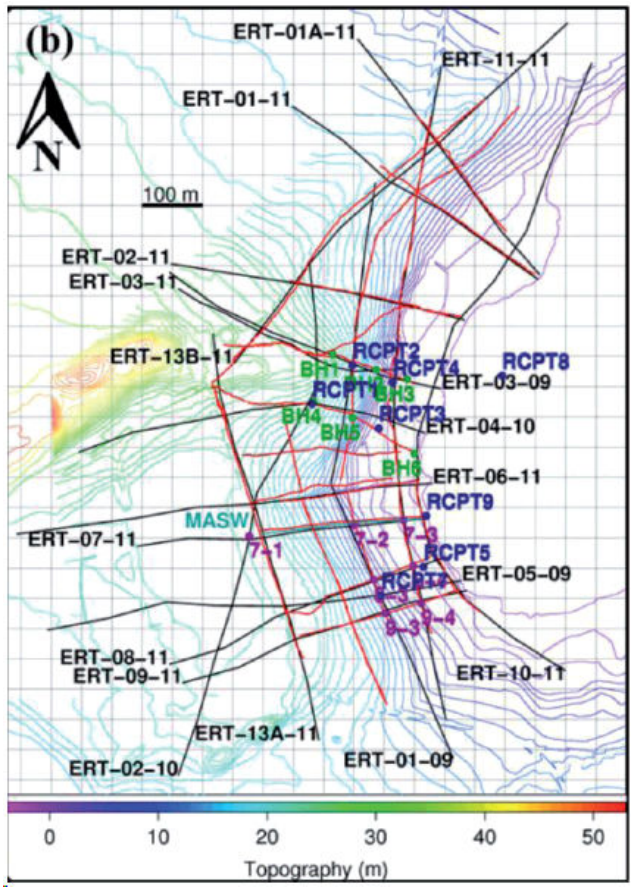
Hoiseth		
MASW, Seismic refraction and ERT	Prosjektnr. 20140622	Kart nr. 24
	Utført JSL	Dato 2015-02-24
	Kontrollert	Godkjent
	NGI	



Målestokk (A4): 1:5 000 Kartprojeksjon: UTM_Zone_33N

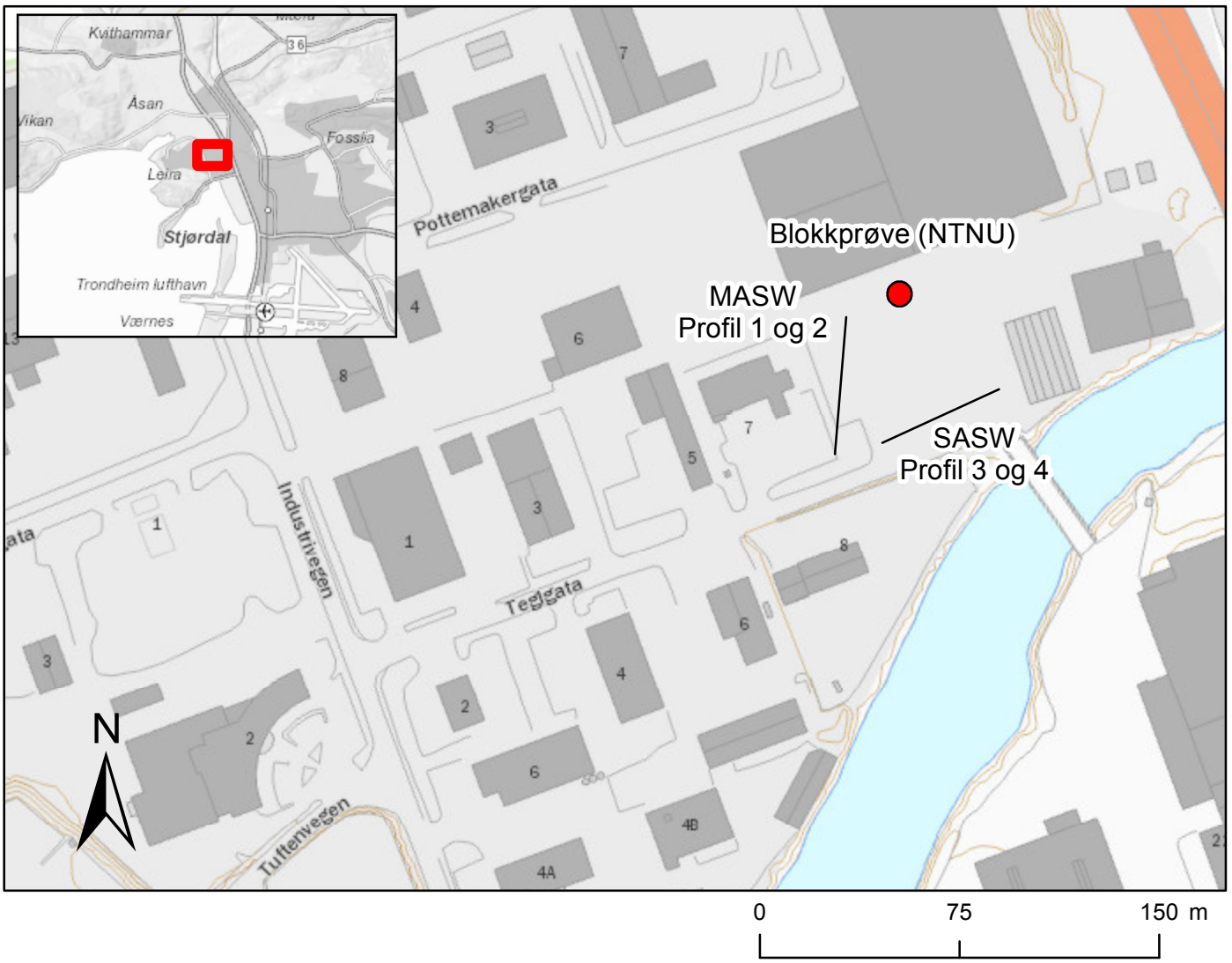
Okstad		
MASW, Seismic refraction and ERT	Prosjektnr. **20140622**	Kart nr. **25**
	Utført **JSL**	Dato **2015-02-24**
	Kontrollert	Godkjent
	NGI	





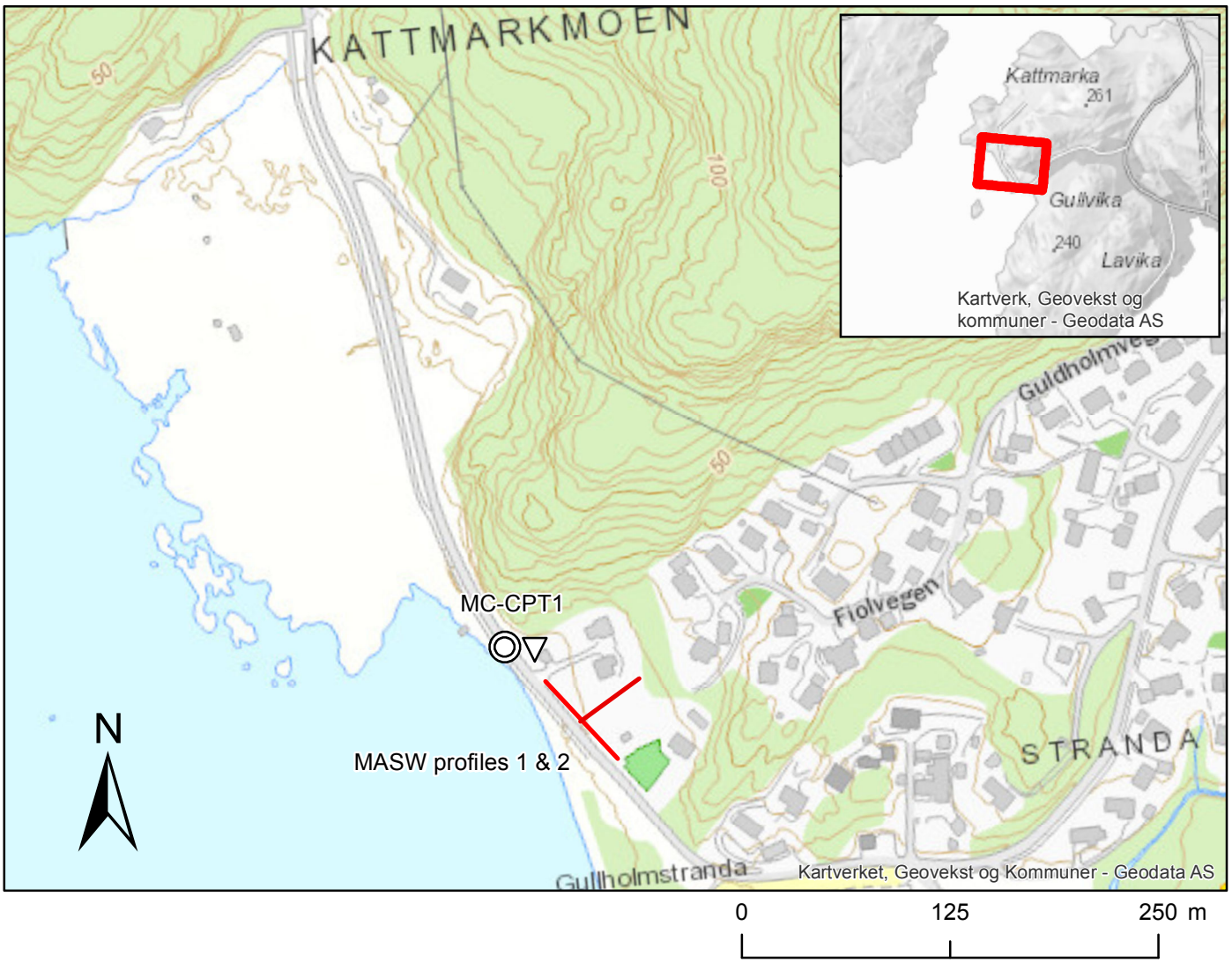
Målestokk (A4): 1:5 000 Kartprojeksjon: UTM_Zone_33N

Rissa		
Forsøksfelt - MASW, ERT, sondering og prøvetaking	Prosjektnr. 20140622	Kart nr. 26
	Utført JSL	Dato 2015-02-24
	Kontrollert	Godkjent
	NGI	



Målestokk (A4): 1:2 500 Kartprojeksjon: UTM_Zone_33N

Glava, Stjørdal		
Forsøksfelt - MASW, SASW, blokkprøver	Prosjektnr. **20140622**	Kart nr. **27**
	Utført **JSL**	Dato **2015-02-24**
	Kontrollert	Godkjent



Målestokk (A4): 1:4 000 Kartprojeksjon: UTM_Zone_33N

Namsos, Kattmarka		
MASW, CPTU, Prøvetaking	Prosjektnr. **20140622**	Kart nr. **28**
	Utført **JSL**	Dato **2015-02-24**
	Kontrollert	Godkjent
	NGI	

Appendix B

SITE DESCRIPTION

Contents

B1 Sites where comparative data are available

In this section data will be presented for sites where comparative V_s data is available, i.e. for MASW and from another method where V_s was directly measured. Sites will be dealt with area by area in Norway in the same order as given in Table 1 of the main report. A summary of the comparisons is given in Table B - 1. In general, the different techniques used for measuring *in situ* V_s gave comparable results (Table B - 1). The most widely used technique in this study (i.e. MASW) has shown to be reliable up to depths of 15-20 m. In order to obtain the most reliable data with this technique, it is strongly recommended that geotechnical engineers and geophysicists join forces prior to data acquisition and during data processing to insure reliable results of V_s.

Table B - 1: Summary of sites where comparative data available

No.	Site	Comparison between	Finding	MASW reliable to depth (m)	Max Vs in reliable data (m/s)
1	Onsøy	MASW and SCPT	Results very similar	16.2	130
7	NGI car park	MASW and SASW	Results very similar	14.8	210
8	Museumpark	MASW and SCPT and cross-hole	Results the same	8.4	135
		MASW and Raleigh wave	Results fit theoretical relationship $V_r = 0.955\ V_s$		
9	Lierstranda	MASW and Raleigh wave	MASW slightly greater than Raleigh wave	12	180
10	Hvittingfoss	SW inversion (MASW), seismic reflection and SCPT	Very good agreement between all techniques	10	200
16	Tiller	MASW, SASW, cross-hole (one depth and SCPTU	Results very similar	19.6	200
17	Berg	MASW and cross-hole	Results very similar	8.2	200
18	Eberg	SASW and seismic refraction	Results reasonably similar	n/a	n/a
19	Esp	MASW, SCPTU and cross-hole (one depth)	Results compare well	21.5	220
20	Klett (South)	MASW and SCPTU	Results very similar	25	260
27	Glava	MASW and SASW	Results compare well	12.5	230
28	Bothkennar	MASW, cross-hole, CSW, SCPT, SDMT	Results more or less identical	13.4	140

B1.1 Onsøy

The site comprises a deep deposit of homogenous soft clay and has been used extensively by NGI and others for research purposes. Two different locations in the same area have been used over the years. These are called Onsøy 1 and Onsøy 2 by Karlsrud and Hernandez-Martinez (2013). The Onsøy 2 site has in general been used since 2000. Comparison of the index properties of the soils confirms that for all practical purposes the clay is identical at both locations. See Appendix A for site location.

A 2D MASW profile across the Onsøy 2 site is shown on Figure B - 1. This confirms the high degree of uniformity of the deposit.

SCPT tests carried out at Onsøy 1 are compared with MASW profiles from both Onsøy 1 (called MASW- S) and Onsøy 2 on Figure B - 1. It can be seen that for all practical purposes SCPT and MASW tests give the same results. MASW data is reliable to about 16 m.

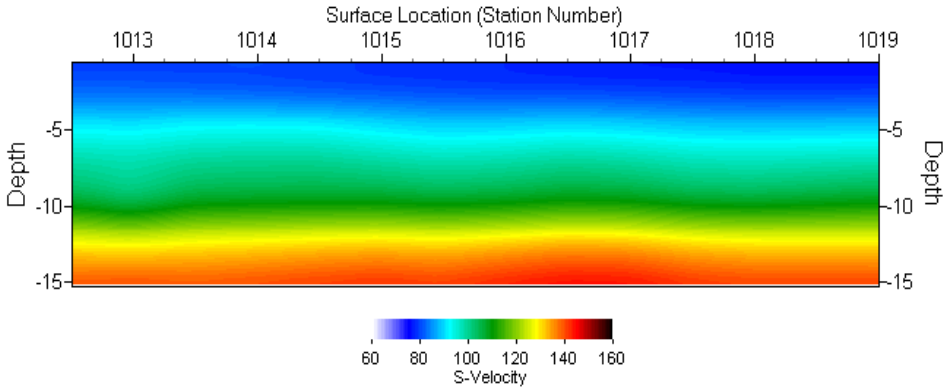


Figure B - 1: Onsøy – 2D MASW profile

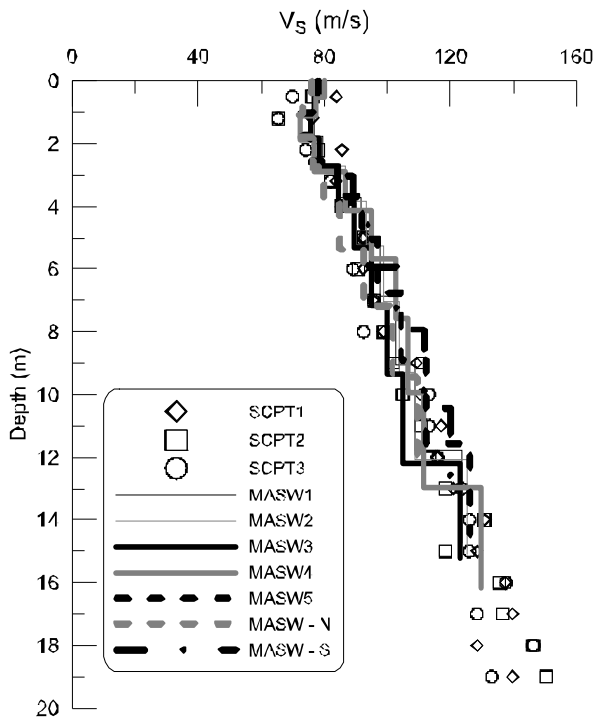


Figure B - 2: Onsøy results (SCPT and MASW-S are from Onsøy 1 site. All other data from Onsøy 2).

B1.2 Oslo NGI car park

The NGI car park has been occasionally used for trials of various equipment, e.g. CPT. V_s was measured using the SASW technique at a site just west of the NGI car park, along the northern boundary of Sogn Kolonihage. See Appendix A for site location. MASW was carried out along the northern border of the NGI car park, perhaps 50 m away from the SASW profile. Some investigation boreholes for works on the adjacent Oslo Ring Road are available, see Figure B - 3. About 8 m of sandy silty clay overlies soft clay, which contains pockets of quick clay. The MASW and SASW results, shown on Figure B - 3, give very similar V_s values. The low SASW V_s values between 2 m and 3.8 m may be due to a data resolution error.

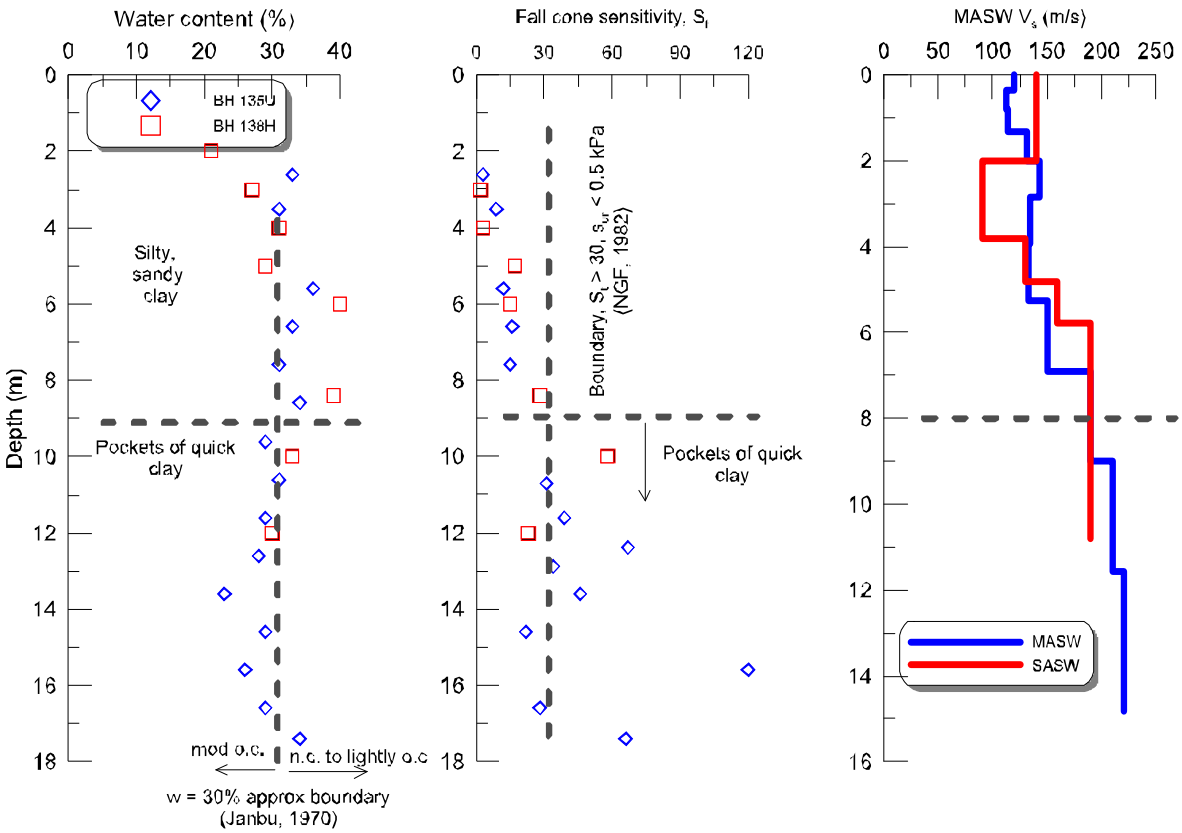


Figure B - 3: NGI car park results

B1.3 Danviksgata and Museumpark Drammen

Similar to Onsøy extensive research has been carried out on the properties of Drammen clay by NGI since the early 1950's. Two adjacent Drammen clay sites were surveyed, i.e. those located close to the city centre at Danviksgata (Profile 1 and 2) and

Museumpark (Profile 3 and 4), see Figure B - 4. Over the top 12 m (zone of interest here) the area is underlain by plastic Drammen clay, see Table 2 of main report.

MASW data from the two sites is shown on Figure B - 4. It can be seen that there is very good consistency between the two MASW profiles at each location. V_s value for Danviksgata are slightly lower than those at Museumpark over the top 4 m to 5 m but below this the results are more or less identical. Below 8 m to 10 m the resolution of the recorded data is somewhat lower than for the shallower zone. MASW tests results together with the other available data are summarised on Figure B - 5. There is generally very good agreement between the MASW values, the SCPT results and the cross-hole seismic values (Figure B - 5), particularly over the top 8 m. For all practical purposes the values given by the various methods are the same.

Raleigh wave velocity (V_r) values derived from Raleigh wave tests show slightly lower values than V_s obtained from MASW, SCPT or cross-hole testing (Figure 4.5b). Theoretically, for an elastic solid with Poisson's ratio equal to 0.5, V_r = 0.955 V_s (Achenbach, 1975) and the data here again seem to fit well with this relationship.

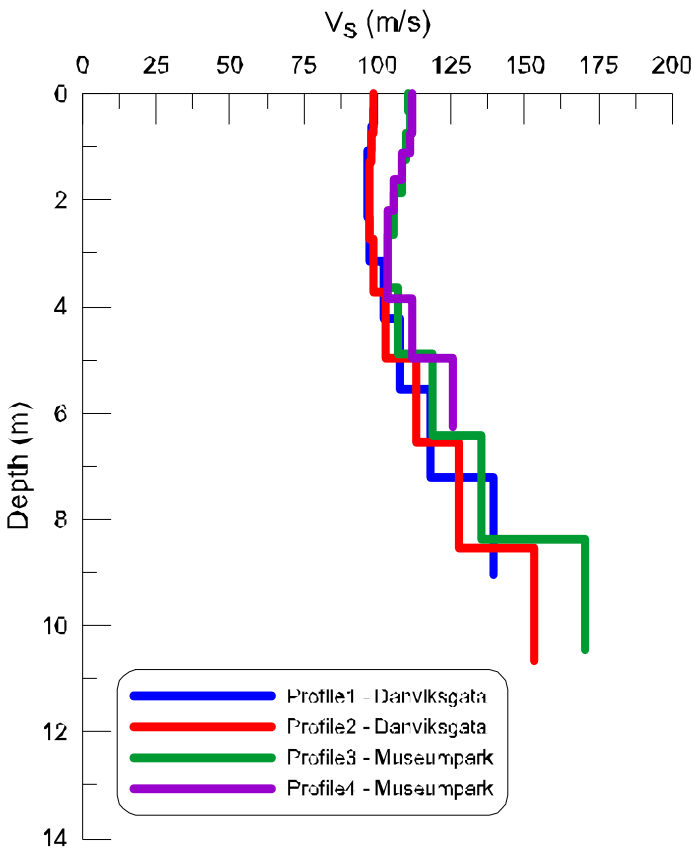


Figure B - 4: Drammen Danviksgata and Museumpark sites

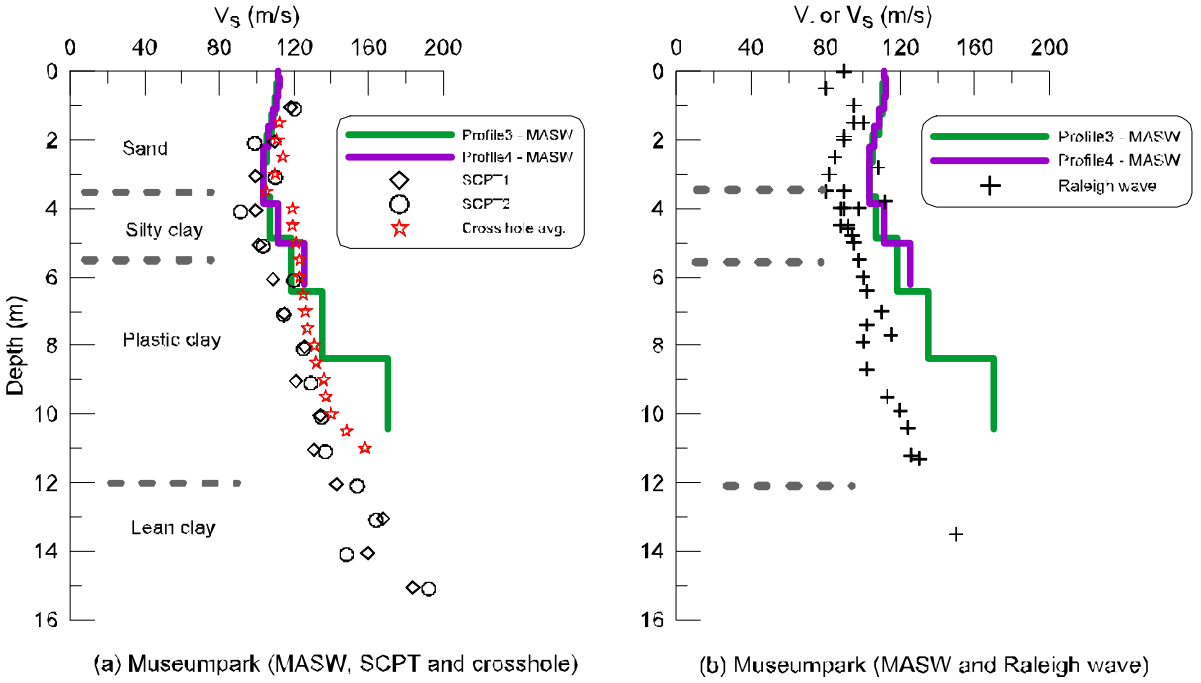


Figure B - 5: Museumpark site (a) MASW, SCPT and cross-hole and (b) MASW and Raleigh wave

B1.4 Lierstranda

The Lierstranda research site, located just east of Drammen town centre, is also a well know NGI research area. It has been used mainly for research into sample disturbance effects into low plasticity clays but MASW and Raleigh wave data are also available for the site as shown on Figure B - 6. As for Museumpark the MASW V_s values are somewhat greater than the V_r measurements. Even allowing for the adjustment according to the $V_r = 0.955\ V_s$ relationship the value remain lower especially between 6 m and 10 m depth.

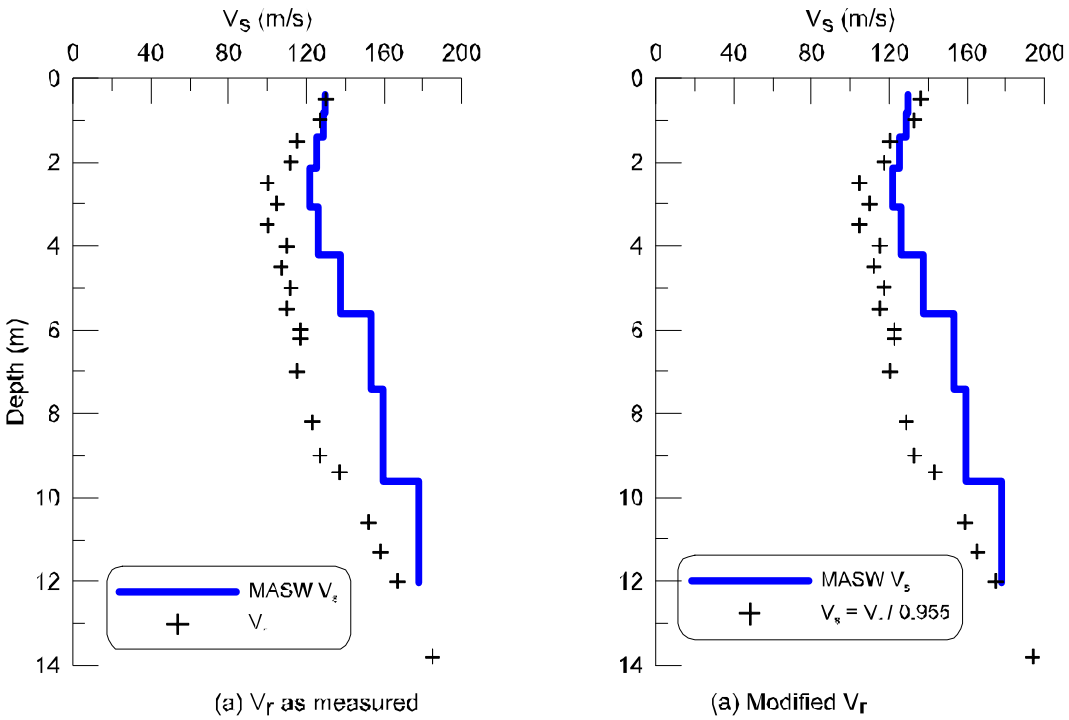


Figure B - 6: Lierstranda site

B1.5 Hvittingfoss

The Hvittingfoss site is located within a quick-clay area. Because of river erosion at the foot of the site, the steep slope and the inhabited area nearby, there was a concern about the soil conditions and stability. Hence, in 2008, the site was mitigated to prevent potential quick clay landslide failure. Because of the large amount of geotechnical data available, this site was selected as a field laboratory to evaluate the integrated use of various geophysics techniques for landslide assessment

Data for the site shown on Figure B-7 confirms the presence of quick clay to at least 8.5 m. There is generally very good agreement between the V_s values from SCPTU, surface wave inversion (MASW) and seismic reflection. It is also possible to predict V_s by correlation from CPTU data (see Table 6 in main report). The CPTU based methods also agree well with the measured V_s values. The technique based on q_t and e_0 seems to give slightly better predictions than that based on q_t and B_q or that based on q_{net}, σ_{v0}' and w (as derived in this project). However the latter new technique seems to better reflect the increase in V_s with depth.

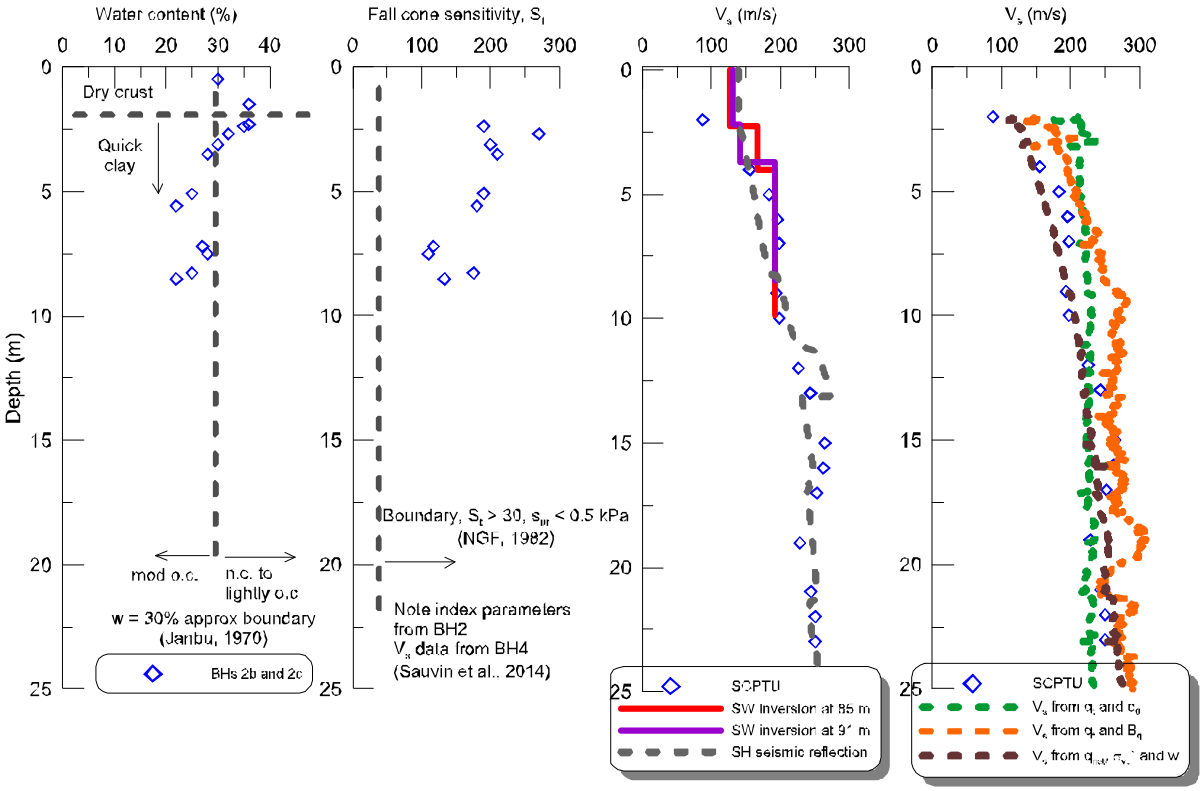


Figure B - 7: Hvittingfoss site

B1.6 Tiller

The Tiller quick clay research site, located just south of Trondheim, has been used by researchers at the Geotechnics Division of the Norwegian University of Science and Technology (NTNU formerly NTH) for research purposes by NTNU for many years. It comprises approximately 8 m of non-sensitive clay over quick clay.

There is good agreement between the MASW and SASW data (latter limited to about 9 m depth), as can be seen on Figure B - 8. This is despite the SASW data being inverted using a relatively simple process. This involved taking the dispersion curve (wavelength versus velocity) and converting wavelength to depth by simply dividing the wavelength by 3 ($\lambda/3$).

There is also excellent agreement between the MASW and SCPTU data. Note that the two SCPTU profiles S5 and S6 were done at the same location as the MASW profiles 5 and 6.

The MASW data also agree very well with the single cross-hole value at 8 m depth.

Data derived empirically from CPTU are also shown on Figure B - 8. In general all three methods match the measured V_s values very well with little difference between the results.

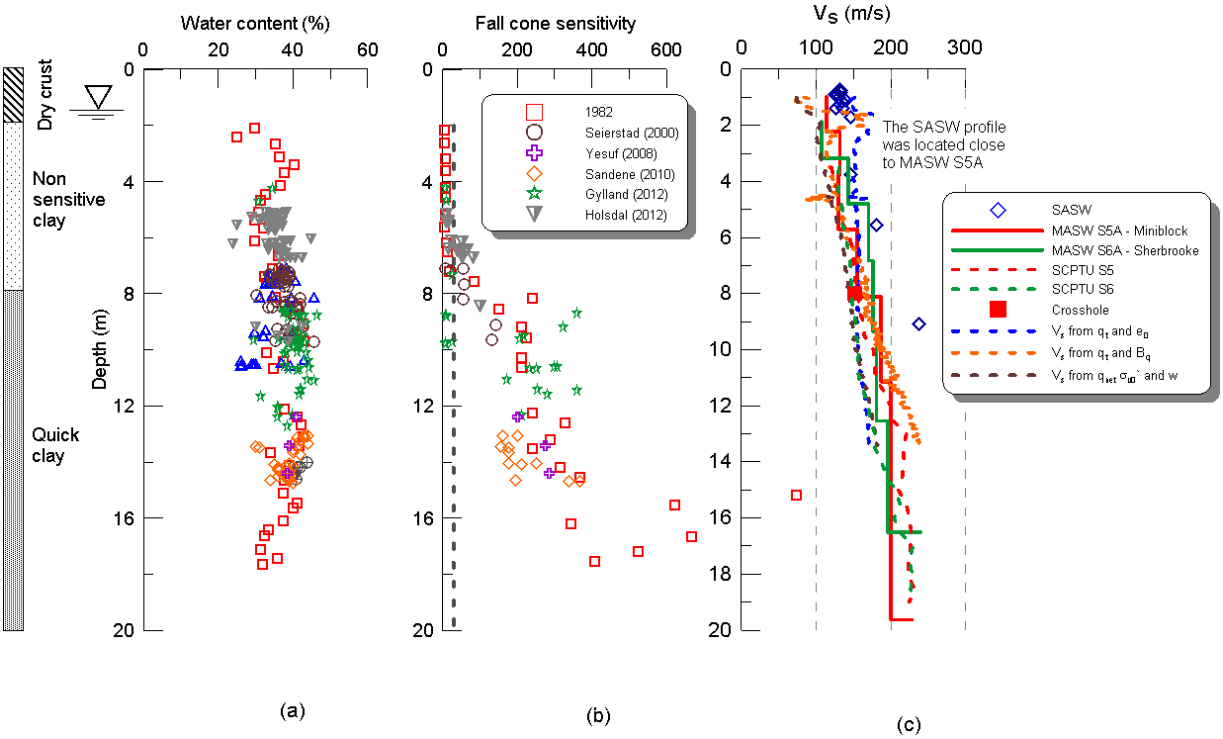


Figure B - 8: Tiller site

B1.7 Barnehage / Berg

These sites are located very close to the NTNU campus at Gløshaugen. They are located on opposite sides of Dybdahlsveg and both sites are located within the zone of the historic Lerkendal quick clay slide (Rømoen, 2006). Index parameters for the two sites (Figure B - 9) show the clay is very similar at the two locations. The results of the cross-hole testing at the Barnehage site and the MASW at Berg generally agree well.

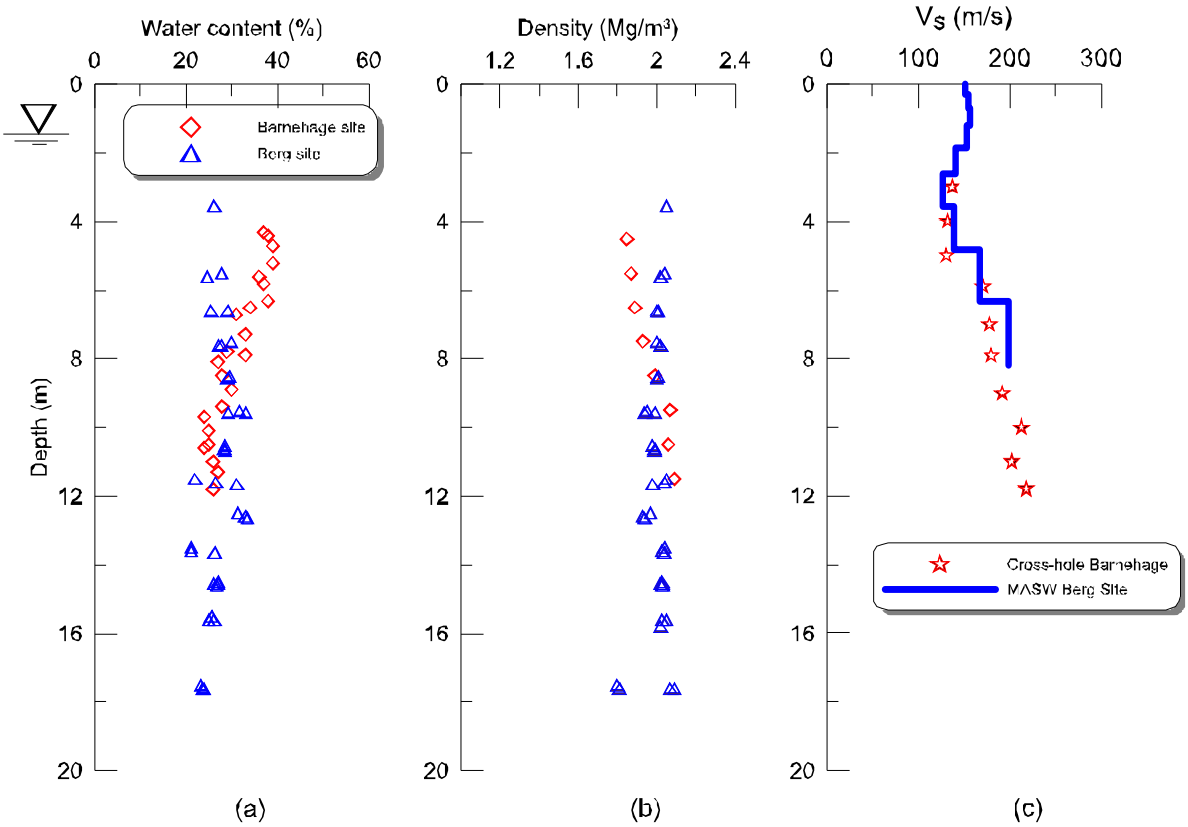


Figure B - 9: Barnehage / Berg results

B1.8 Eberg

Although there is no MASW testing available for this site to date, it is included as the clay is a little unusual for the area being slightly organic with relatively high water content and low density (Figure B - 10). The Eberg site was also used for research purposes by NTNU for many years. SASW data is compared to V_s determined from a simple seismic refraction study on Figure 4.10. For the latter V_s was calculated from V_p assuming a Poisson ratio of 0.49.

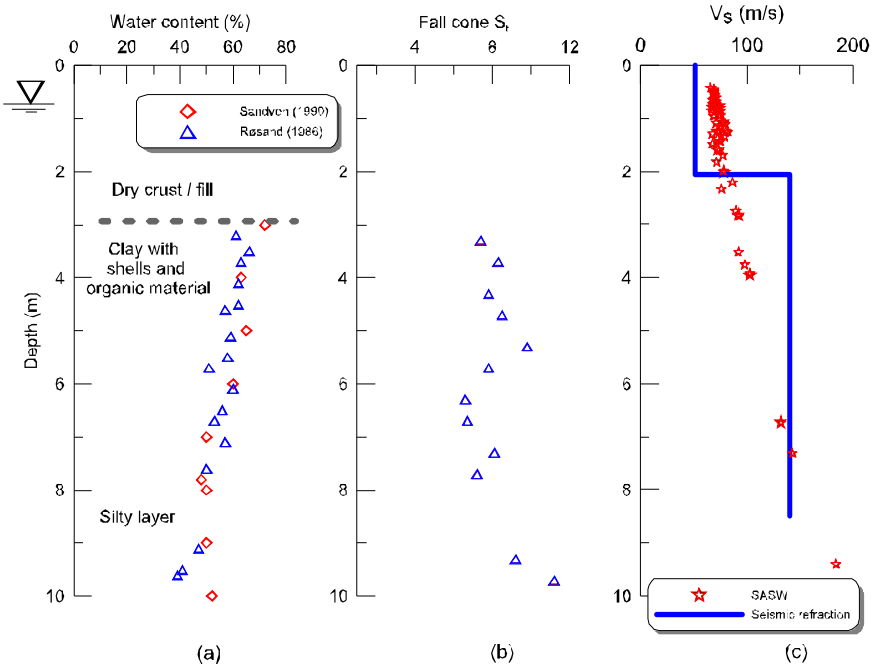


Figure B - 10: Eberg results

Despite the simplicity of the seismic refraction test and the relatively crude nature of the SASW inversion (see comments on Tiller site above), the two sets of data compare reasonably well.

B1.9 Esp

The Esp research site has been relatively recently been developed following the landslide there in 2012. It is underlain by both non-sensitive and quick clay, see

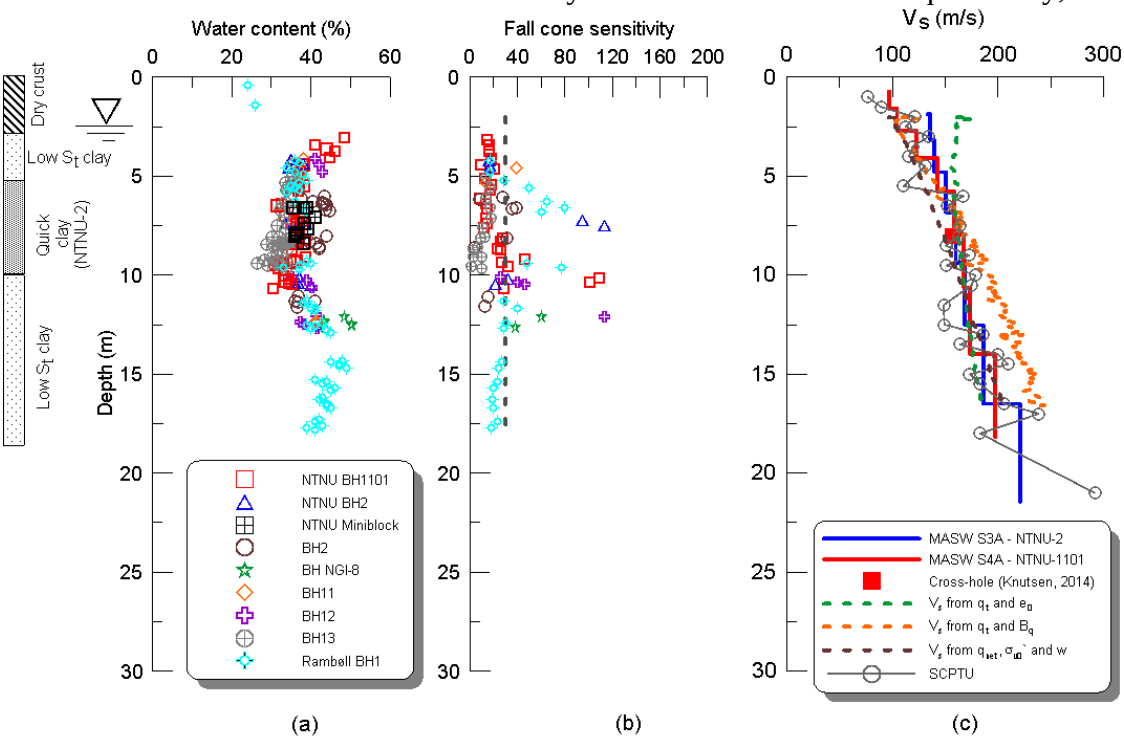


Figure B - 11. As the site is in development the depth range over which testing is available is so far relatively limited. However available testing shows very good agreement between MASW, SCPTU and cross-hole (tests at 1 depth only). The SCPTU V_s values below 15 m are somewhat scattered.

Similar to the discussion above it is possible to compare the measured V_s values with those predicted from CPTU results as shown on Figure B - 11. Again the measured and predicted values agree very well. Arguable the relationship based on q_{net}, σ_{v0}' and w performs best as it gives values very close to those measured and also captures well the profile of increasing V_s with depth.

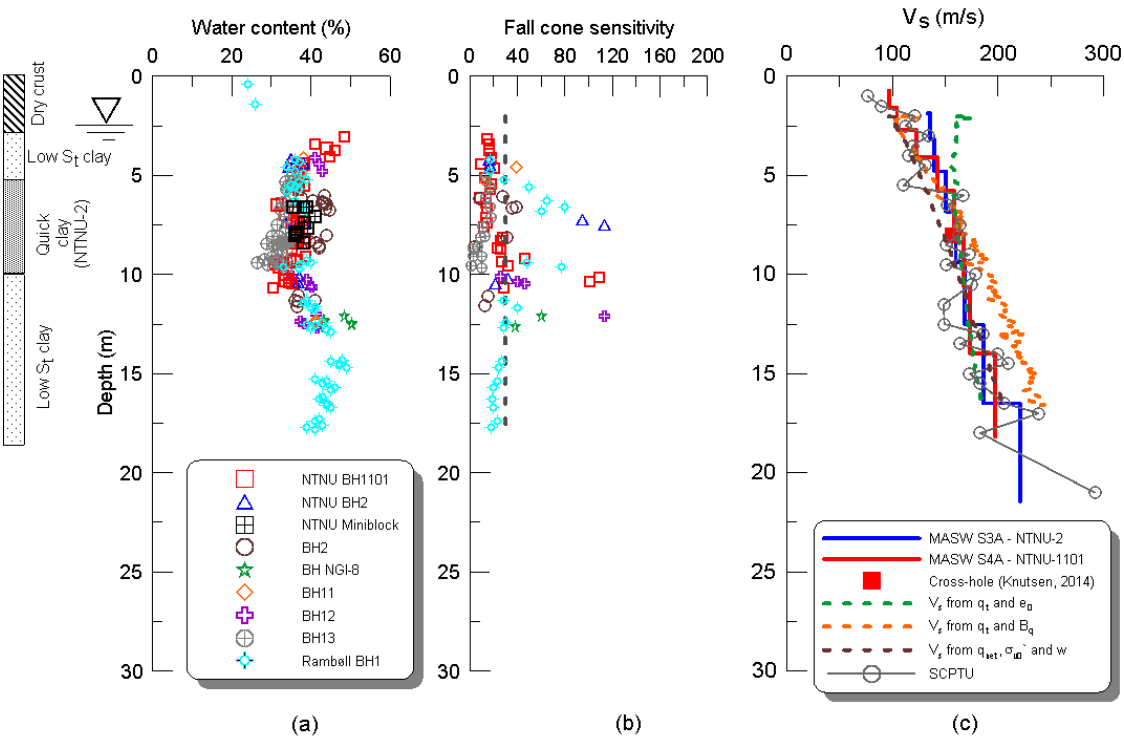


Figure B - 11: Esp results

B1.10 Klett

The Klett research site was developed by the SVV, NGI, Multiconsult in conjunction with the new E6 developments. Note this site is located to the south of the existing E6. Another research area was subsequently developed to the north of the E6. Several aspects of quick clay behaviour have been inspected including use of RCPTU, improved in situ vane testing, use of lime-cement columns etc. The site comprises non-sensitive clay to about 6-8 m and quick clay below this (actually very similar to Tiller), see Figure B-12. Two MASW profiles are available near to boreholes 1502-1503. Results show only a slight increase *in situ* V_s with depth. The SCPTU and MASW results agree very well.

MASW results are also compared through those obtain from correlation with CPTU data through several empirical relationships in Figure B-12. The relationships show comparable results to in situ MASW results. Arguably the relationship between V_s and q_{net}, σ_{v0}' and w performs best as it gives values closest to those measured.

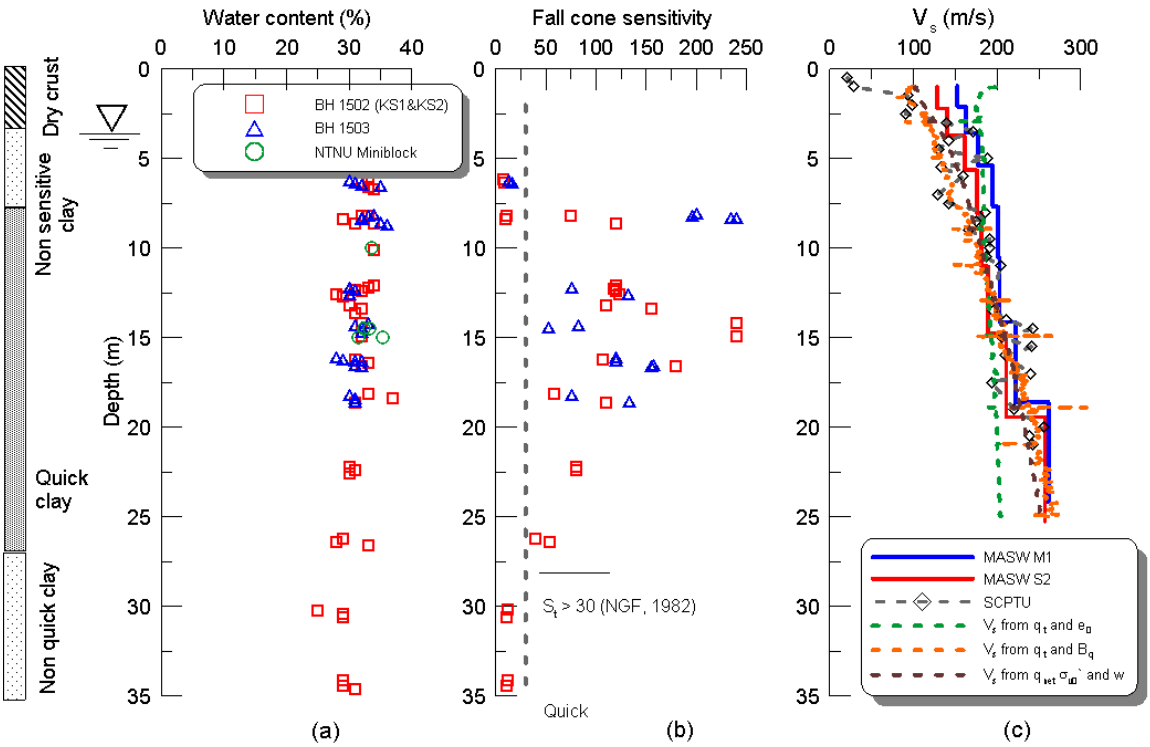


Figure B - 12: Klett (South) results with in situ V_s and V_s from CPTU correlations.

B1.11 Glava

Glava clay has been investigated since the mid 1980's. Unfortunately the specific site under study here is no longer available. Four MASW profiles were taken and the results show a very high degree of consistency (Figure B - 13). Below 10 m to 12 m the data shows lower resolution. SASW values for depths down to about 7m show values which are very similar to the MASW V_s. This is despite the SASW inversion process being relatively simple (see comments on Tiller site above).

V_s predicted from CPTU matches well with the measured data to about 8 m and then is lower than the MASW V_s below this depth. The techniques based on q_t and B_q and on q_{net}, σ_{v0}' and w give very similar results.

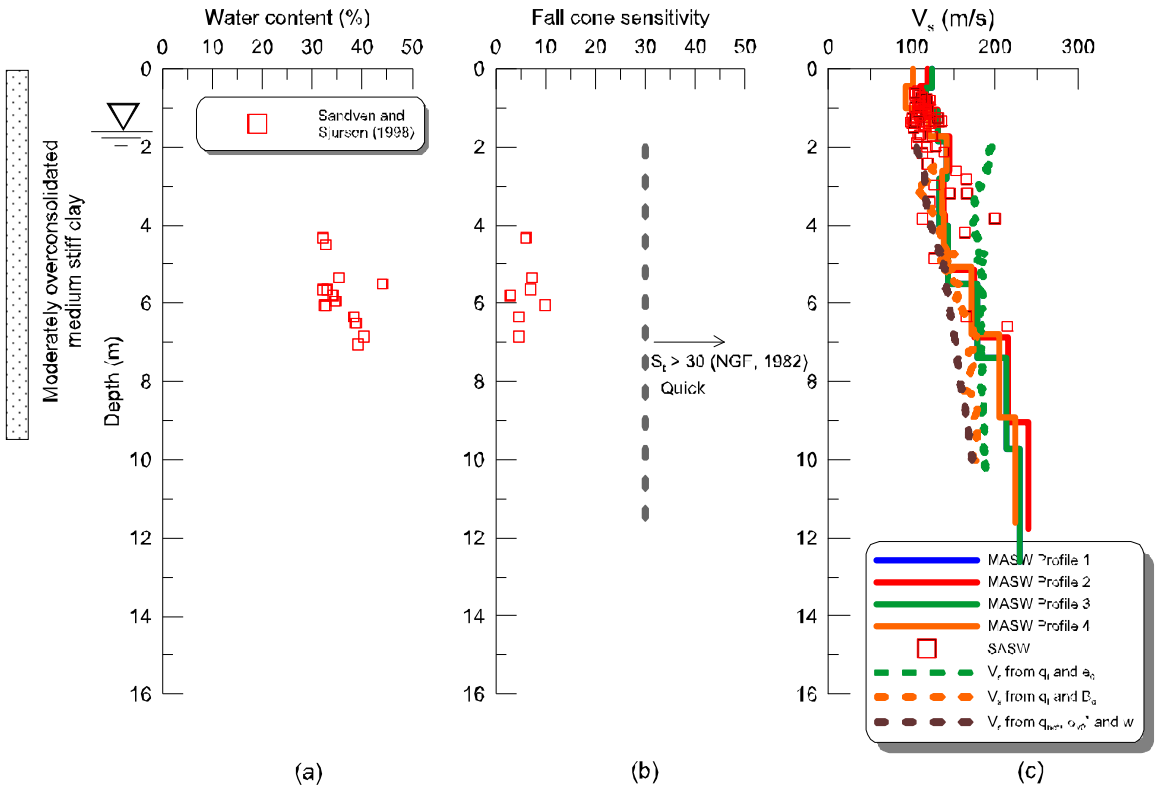


Figure B - 13: Glava results

B1.12 Scotland - Bothkennar

The UK soft clay research site at Bothkennar has been very well characterised, see Géotechnique, 42, No. 2, 1992. It is included in this study as NGI have carried out some testing on block samples from the site and have included the results in various NGI databases, e.g. Karlsrud and Hernandez-Martinez (2013). In addition shear wave velocity has been measured at the site using various techniques and hence there is a good database with which to compare the MASW results. At least 5 investigations have been carried out at the Bothkennar research site as follows:

- University of North Wales (Hepton, 1988): seismic cone (SCPT) and seismic dilatometer (SDMT).
- UK Building Research Establishment (BRE) (Powell, 2001; Powell and Butcher, 1991): cross-hole and SCPT
- Surrey University (SU) (Hope et al., 1999; Sutton, 1999): cross-hole
- GDS Instruments Ltd. (Sutton, 1999): continuous surface wave (CSW)
- University College Dublin (UCD) (Long et al., 2008): MASW

The results shown on Figure B - 14 show excellent agreement between MASW V_s and V_{hh} and V_{hv} (crosshole) and V_{vh} (seismic CPT). These data also confirm that the site has a very low degree of anisotropy of small strain stiffness.

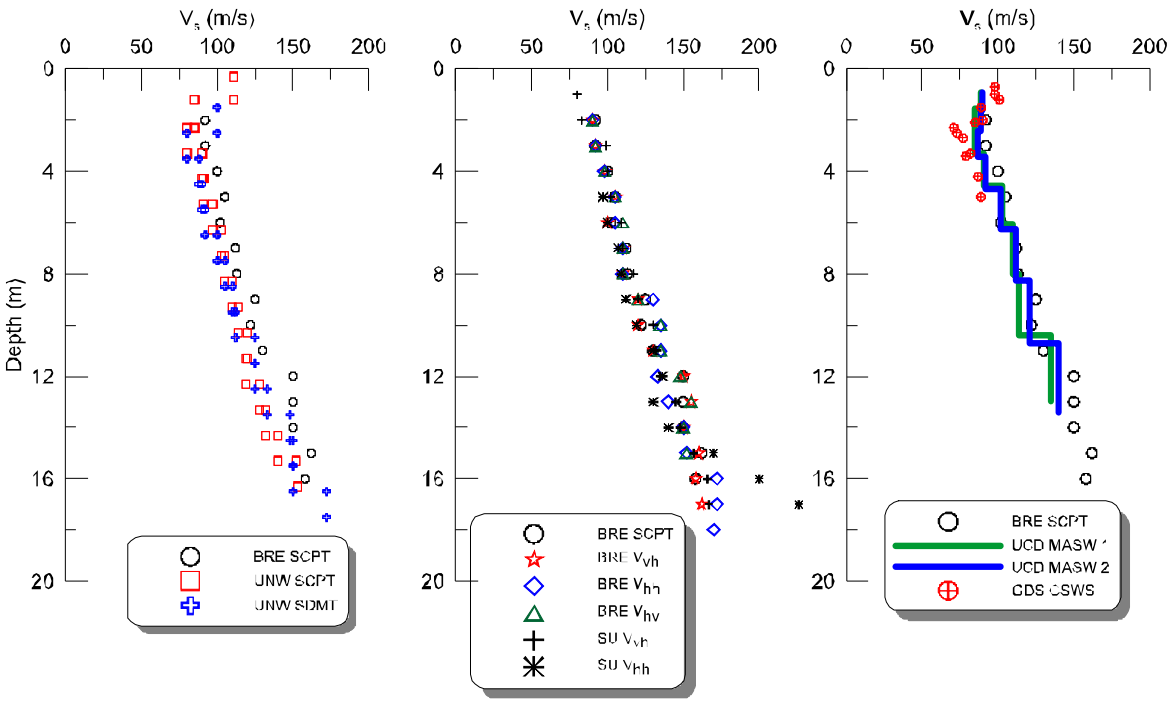


Figure B - 14: Bothkennar results

B2 Sites where no comparative V_s data available

B2.1 Seut Bridge Fredrikstad

An MASW survey was carried out at the Statens Vegvesen / Multiconsult research site near the Seut Bru at Fredrikstad on 14 June 2012. The overall project comprises the improvement of the Rv100 highway at Simo Ørebekk. Work was carried out on two areas located east and west of the Seut Bridge as can be seen on Figure B - 15. Both locations are underlain by a deep deposit of soft, slightly organic and quick clay. MASW results are similar at both locations and seem reliable to 10 m.

This is confirmed by correlations with CPTU as shown on Figure B - 16 below. Here data for Line 1 (west) is used as an example. All three correlations work reasonably well. As was found for the Glava site above the correlations between between V_s and q_t/B_q and V_s and $q_{net}/\sigma_{v0}'/w$ seem to work best and are able to capture the increase in V_s with depth.

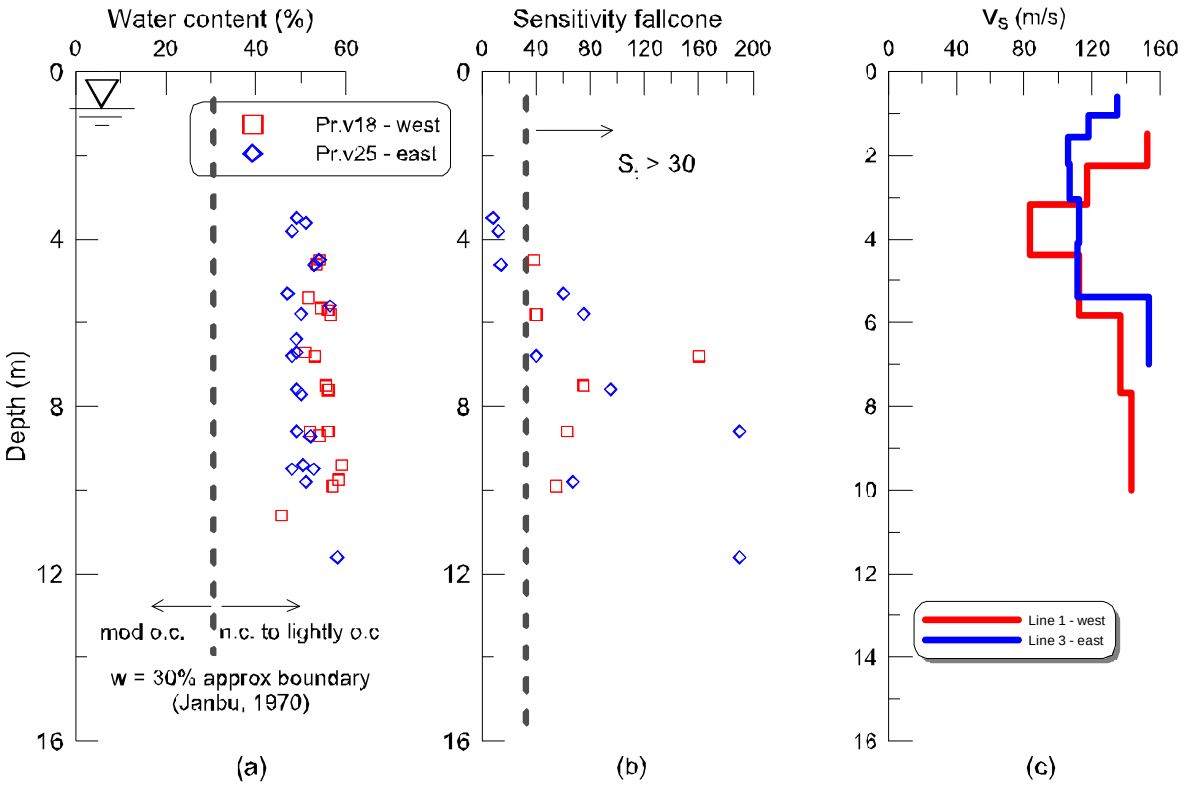


Figure B - 15: Seut Bridge results

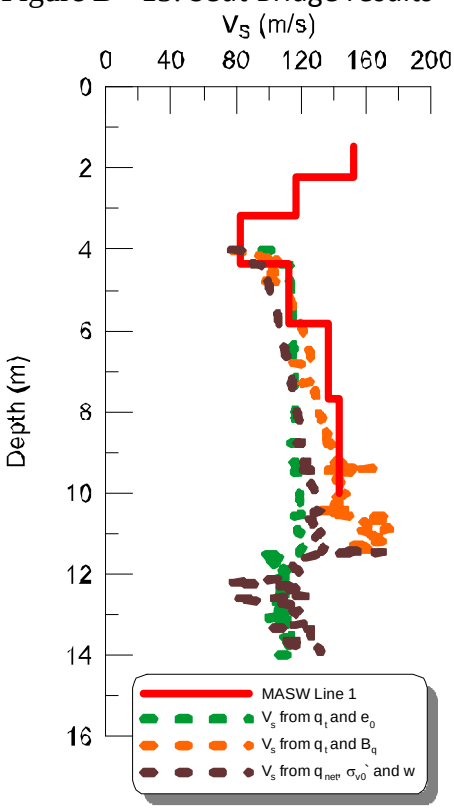


Figure B - 16: Seut Bridge V_s from CPTU

B2.2 Eidsvoll and Hvalsdalen

Work at the Eidsvoll and Hvalsdalen sites was undertaken in conjunction with the high speed rail link between Oslo and Gardermoen airport (Flytoget). Block samples were taken and the results have been included in the various NGI databases. The sites are summarised together here as the soil conditions are very similar, even though they are located some distance from one another. From Figure B - 17 it can be seen that the water content at the Hvalsdalen site is slightly higher and that the clay is more sensitive. Note there is one S_t value = 240 at 15.2 m depth. Results of the MASW surveys at the two sites show very similar results which are consistent with the presence of deep deposits of soft clay. Below about 12 m at both sites the resolution of the data decreases somewhat but it is suggested that the V_s values are reliable to the full measured depth in both cases.

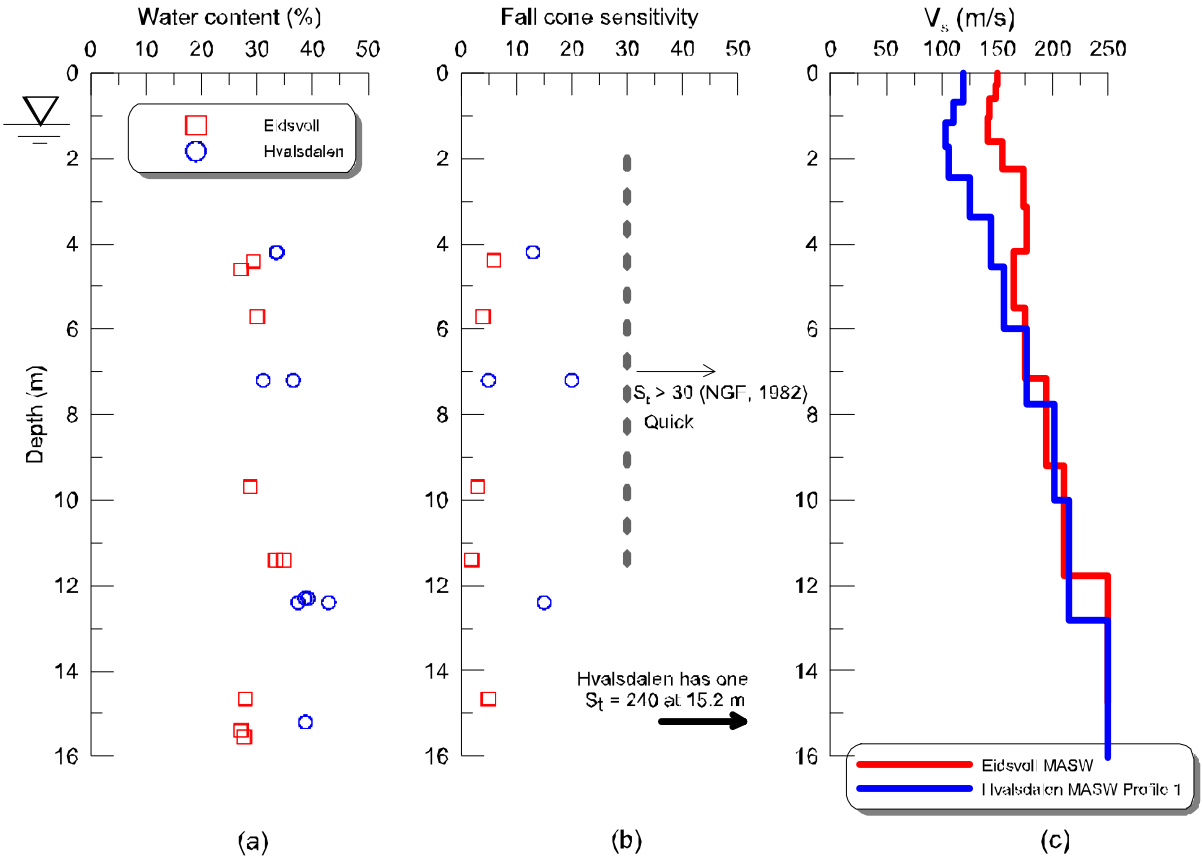


Figure B - 17: Eidsvoll and Hvalsdalen

B2.3 RVII

The RVII (Statens vegvesen research site) is underlain by at least 20 m of soft homogenous clay which becomes highly sensitive / quick with depth. The three V_s profiles indicate consistent results. Below 12 m to 14 m the data shows lower resolution. No independent V_s measurements are available and it is only possible to compare the measured V_s profiles to those derived empirically from CPT, as shown on Figure B - 18. Above 4 m the CPTU q_t values are relatively high, reflecting the desiccated crust and the resulting empirically derived V_s values are much higher than those measured by the MASW surveys. Between 4 m and 12 m the measured and calculated values are very similar. Below 12 m the empirically derived V_s value from the q_t and B_q data is much higher than that derived from the q_t and e_0 or from the q_{net}, σ_{v0}' and w data. Overall the latter is closest to the measured V_s value. This suggests that MASW is reliable to at least 16 m.

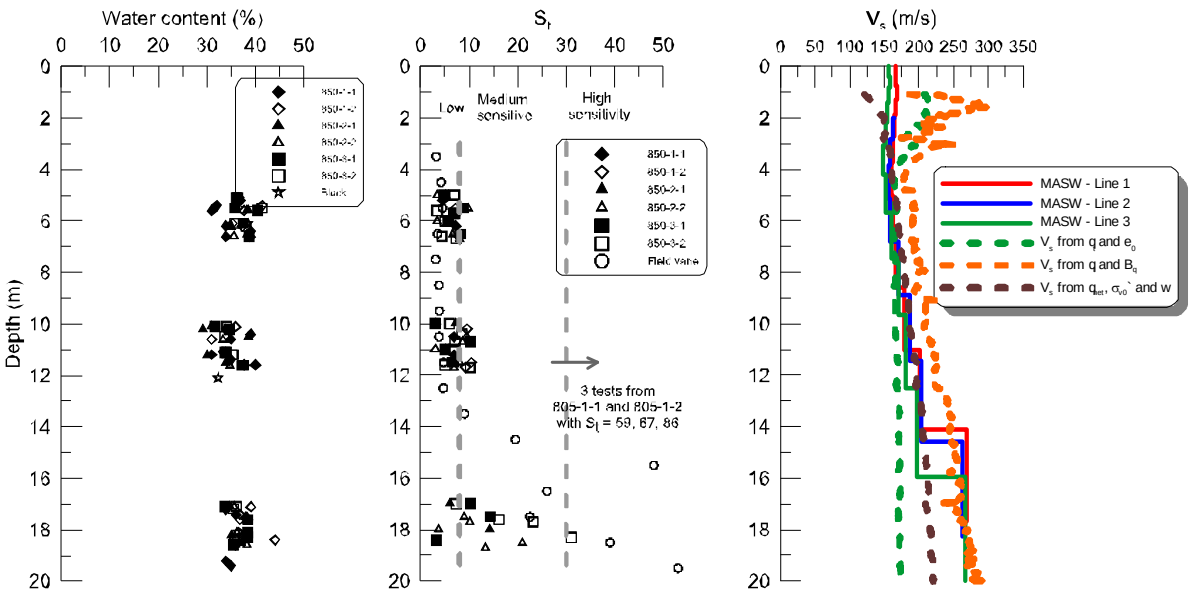


Figure B - 18: RVII results

B2.4 Dobbeltspor Skøyen – Asker, Parsell Sandvika

This is the site of an extension / widening of the main railway line. The site is underlain by a relatively thin dry crust (2 m?) over two distinct clay layers, as shown on Figure B - 19. From 2 m to 5 m the clay is of low sensitivity but beneath this (to 16.5 m proven), the clay is quick. The MASW V_s values seem to be higher and show an increase with depth in the quick layer in comparison to the medium sensitive layer. Nonetheless it would seem that V_s is reliable to the depth measured, i.e. about 10 m.

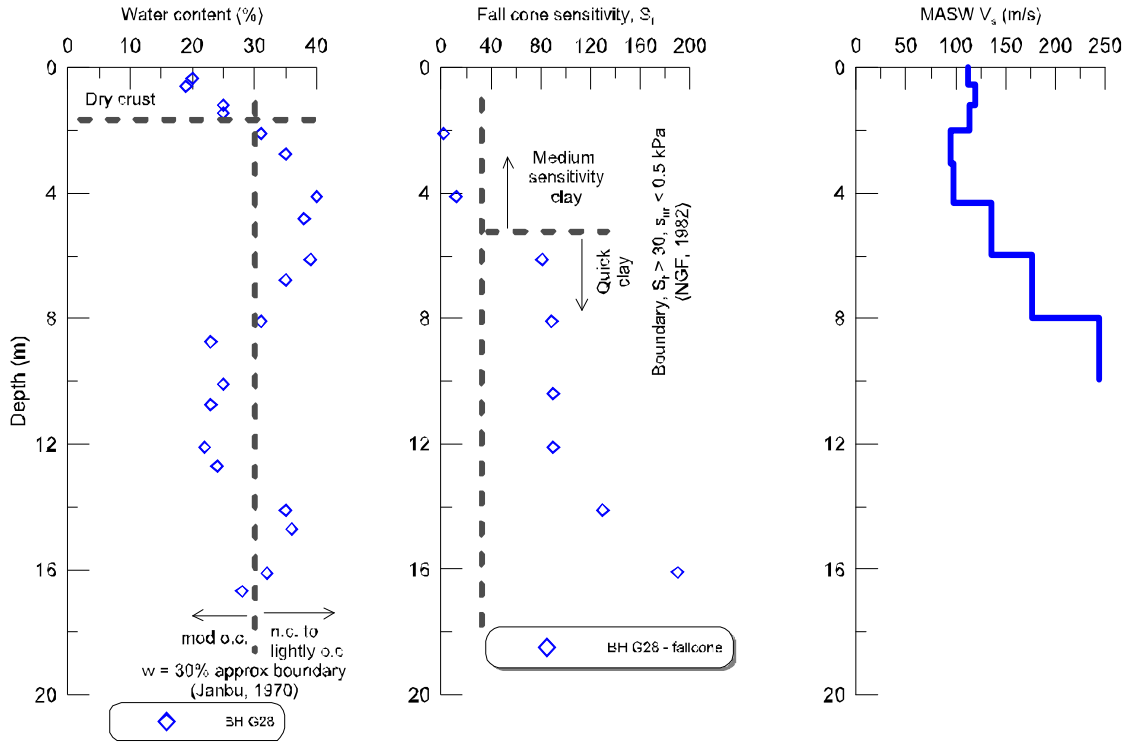


Figure B - 19: Skøyen – Asker

B2.5 Smørgrav

The Smørgrav test site is located just east of the town of Vestfossen. The site has a designated quick clay hazard level of high and has also previously been used by the NGI for quick clay investigations and for the assessment of the use of geophysics to assess landslide susceptibility in a similar manner to the Hvittingfoss site. Data for the site, shown on Figure B-20, shows that it is underlain by soft clay and that there is distinct quick clay later between about 5.5 m and 14.5 m.

MASW profiles, taken in a quick area of the site and a non-quick area show little difference. The MASW V_s profiles can also be compared to the values derived empirically from CPTU. In general there is good agreement between the measured and predicted profiles. All three methods seem to slightly under-predict V_s, especially below 13 m. As has been found for several sites previously the q_t/B_q correlation seems to better capture the increase in V_s with depth.

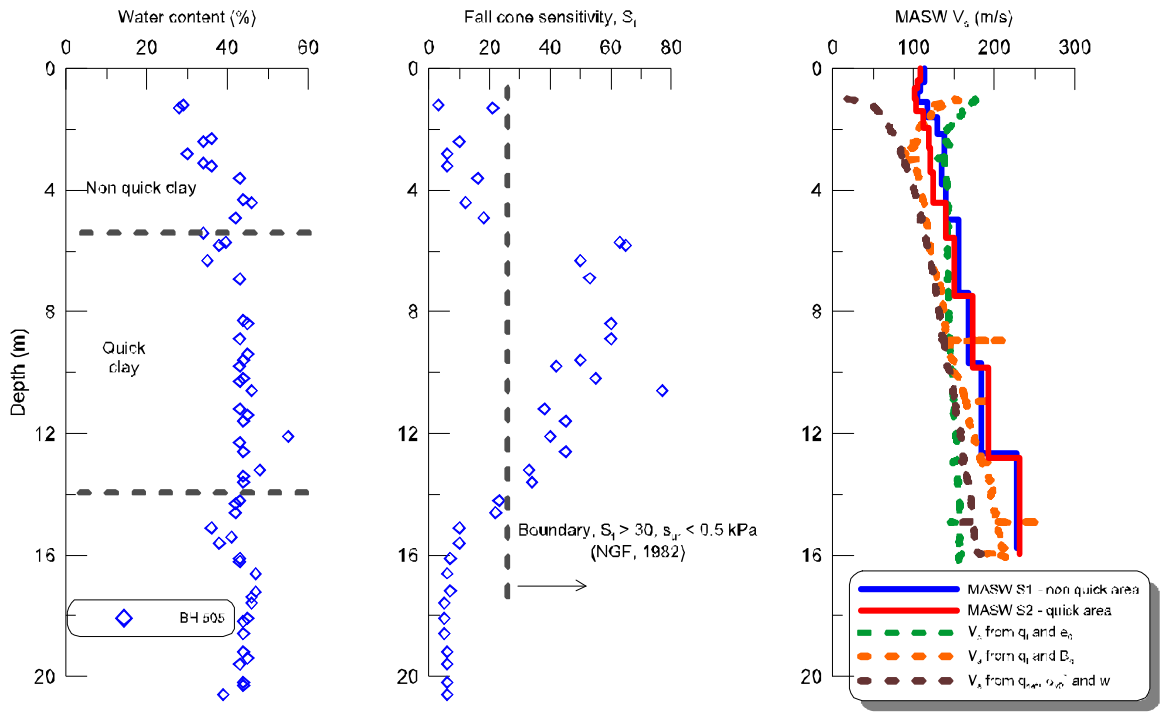


Figure B - 20: Smørgrav data

B2.6 Vålen

The Vålen site is located some 750 m to the north-east of Smørgrav. The area has a documented history of quick-clay landsliding, the most recent event occurring in 1984. Several escarpments are visible in the area surrounding the site. Similar to Smørgrav and Hvittingfoss the site was used to study the use of geophysics in characterising landslide areas. As can be seen from Figure B-21 the ground conditions are very similar to those at Smørgrav with the exception that here the clay is of low sensitivity.

The MASW V_s profile can be compared to the values derived empirically from CPTU (Figure B-21). Once again there is reasonable agreement between the measured and predicted profiles with, as has been found previously for the Glava and Seut Bridge sites for example, the techniques based on q_t/B_q and $q_{net}/\sigma_{v0}'/w$ seem to work best. It can also be concluded that MASW is reliable to the full measured profile of 24.8 m.

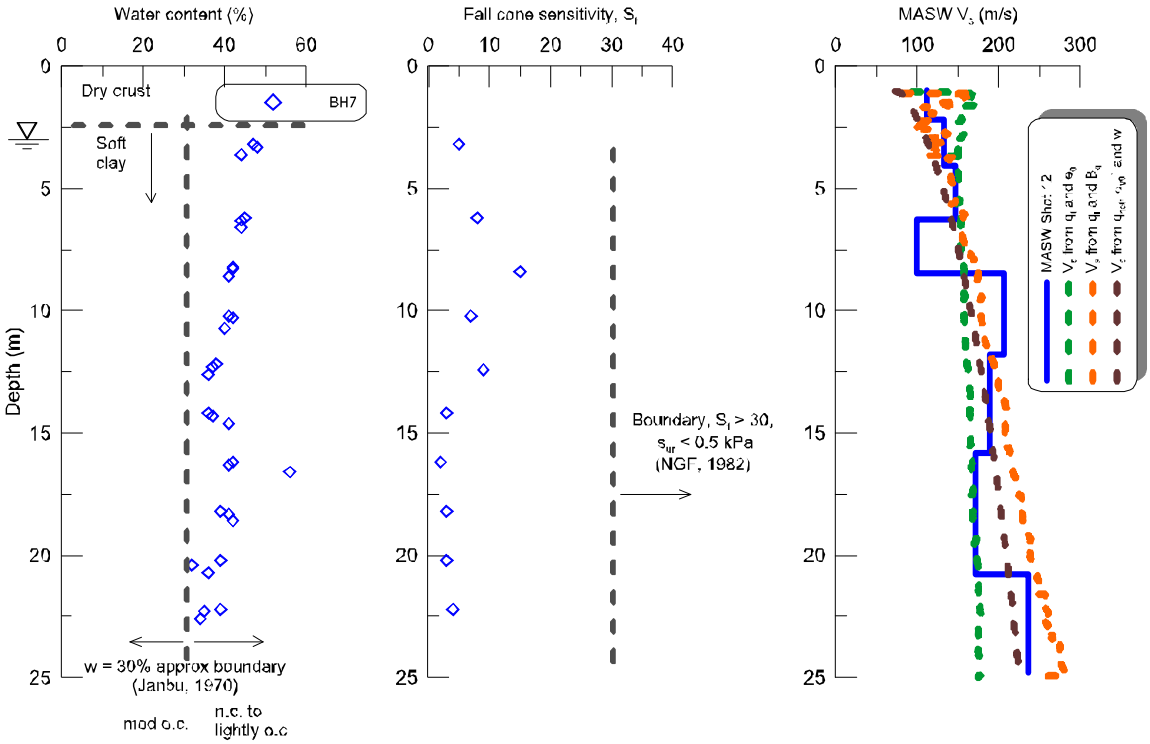


Figure B - 21: Vålen data

B2.7 Farriseidet

This is an ongoing project and is part of the Ny Jerbane Farriseidet – Telemark Grense. As can be seen on Figure B-21, this is a very unusual site for southern Norway. It is underlain by 3 m of peat (von Post H8-H9), over 5 m of quick organic clay over rock at 8 m. The clay has high water content of the order of 90% and has low unit weight of the order of 14.7 kN/m^3. These data are consistent with very low shear wave velocity values of the order of 40 m/s - to 50 m/s.

Resistivity measurements show bedrock to be present at about 8 m. Therefore the V_s values recorded below about 8.5 m need to be treated with caution.

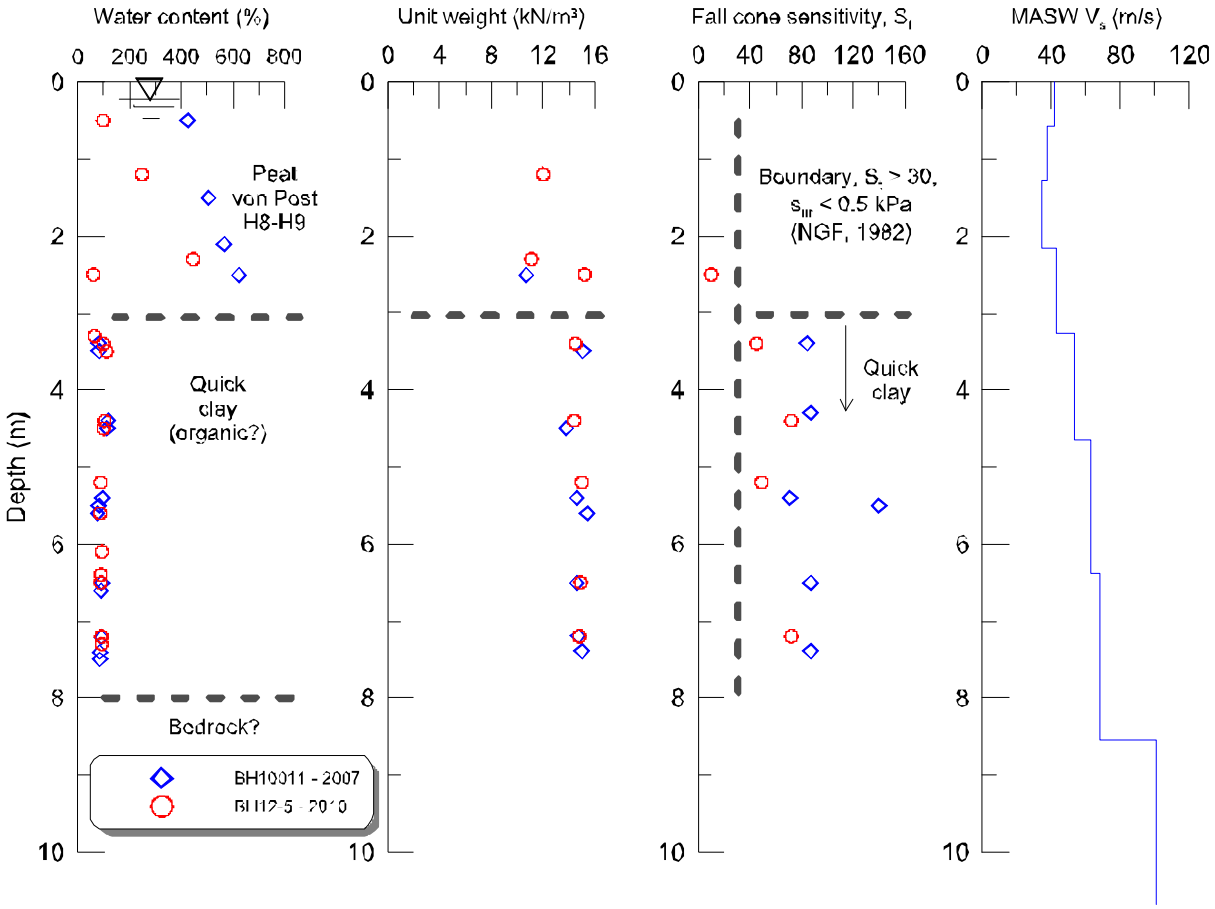


Figure B - 22: Farriseidet data

B2.8 Månejordet

The Norwegian Public Roads Administration (Statens Vegvesen) is proposing to construct a new 4 lane road (E 18) outside Larvik as part of the overall E18 Bommestad – Sky project. In the Månejordet there will be an excavation about 15 m deep in soft quick clay. As part of the ground investigation work MASW testing was also carried out. Data for the site is shown on Figure B.23. The water content data suggest that, relative to the other sites above, the material is reasonably variable. The materials at this site also seem to have lower clay content than those reported above. There is a clear quick clay layer from about 5 m. No parallel V_s measurements or CPTU data are available to compare with the MASW testing. However, except possibly for the jump in the data at 8.2 m it would seem to be reliable data.

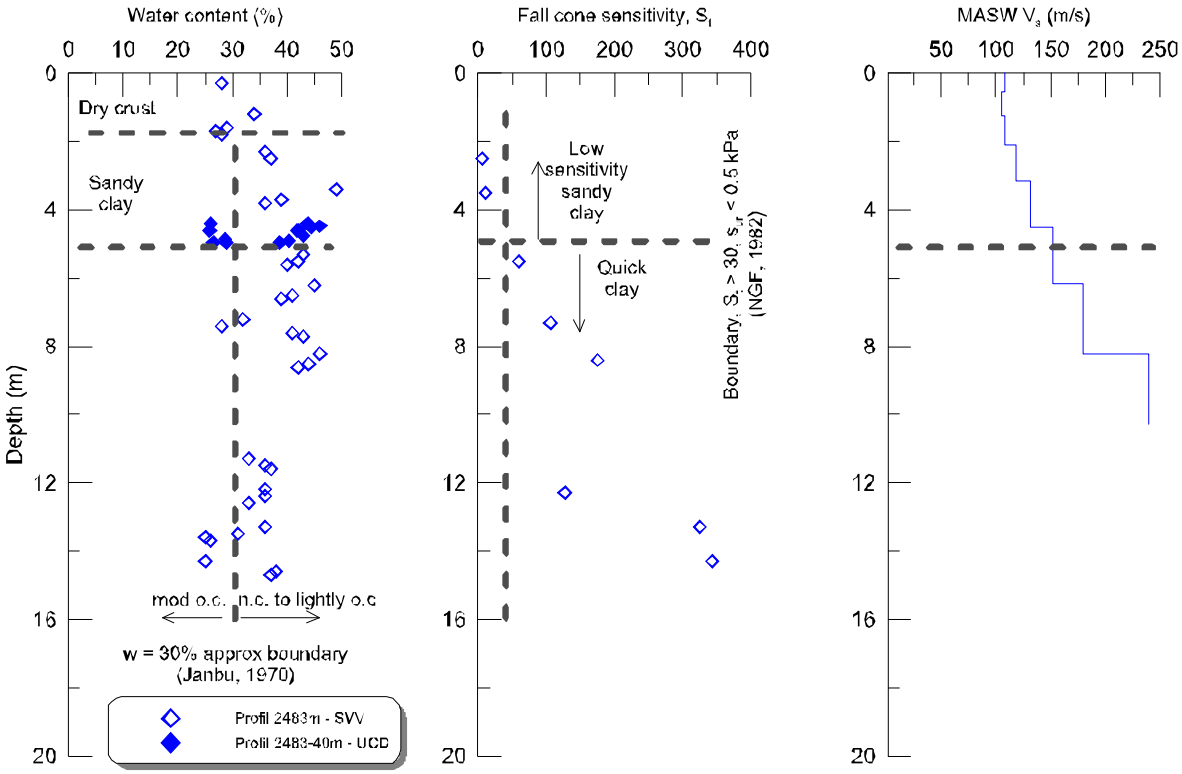


Figure B - 23: Månejordet

B2.9 Skienselven

This site is being studied by NGI for the purposes of designing protection measures for slopes in quick clay. Ground conditions comprise about 6 m of sandy silty clay over quick clay. Again there are no data with which to compare the MASW results but these appear reliable at least until the jump in the V_s values at 7.7 m

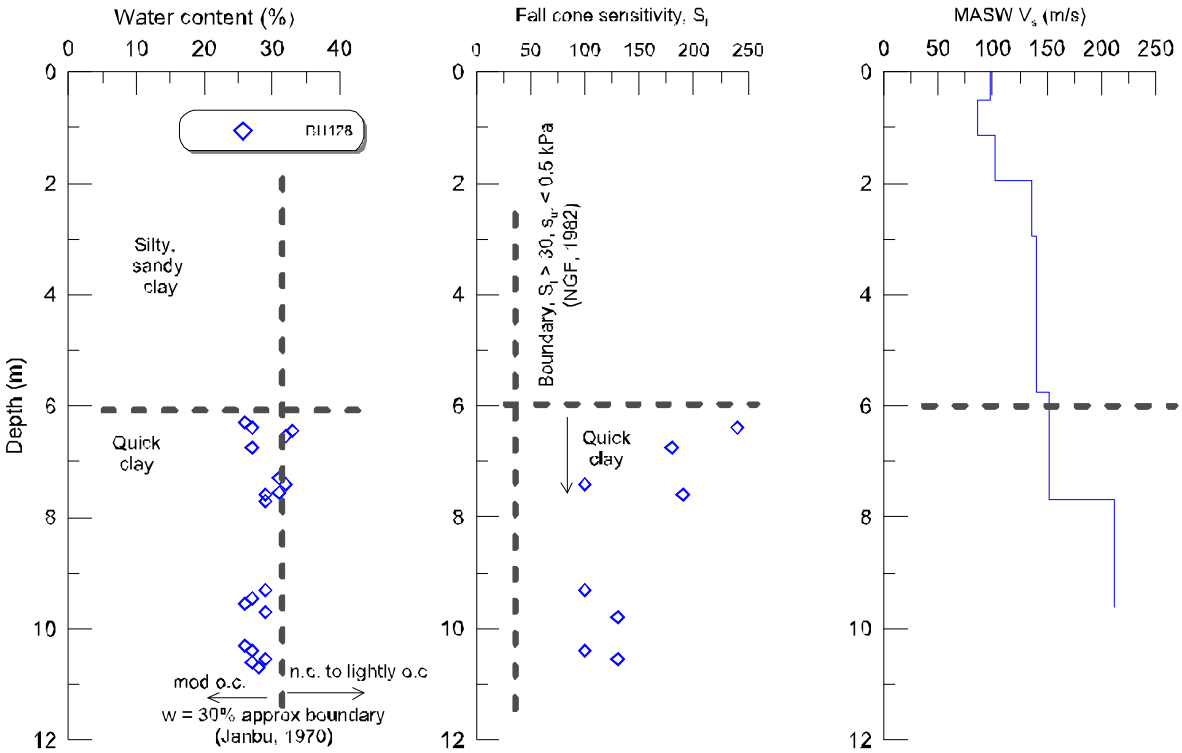


Figure B - 24: Skienselven

B2.10 Dragvoll

The Dragvoll site has largely been developed for research into the use of salt wells to treat quick clay (PhD student Tonje Eide Helle). This particular site is some 200 m south of a previously characterised Dragvoll site (Emdal et al., 2012). The area is characterised by some 4.5 m of non-sensitive clay over quick clay (Figure B-25). The quick clay is very soft and is a difficult material to handle in the laboratory.

MASW profiles S8 and S9, taken outside the salt well area by APEX are very similar. Surface wave inversion work (in essence similar to MASW) gives more or less the same values as the MASW.

V_s values derived from q_t and e_0 agree reasonably well with the measured data but as has been found above do not seem to reflect the increase in V_s with depth.

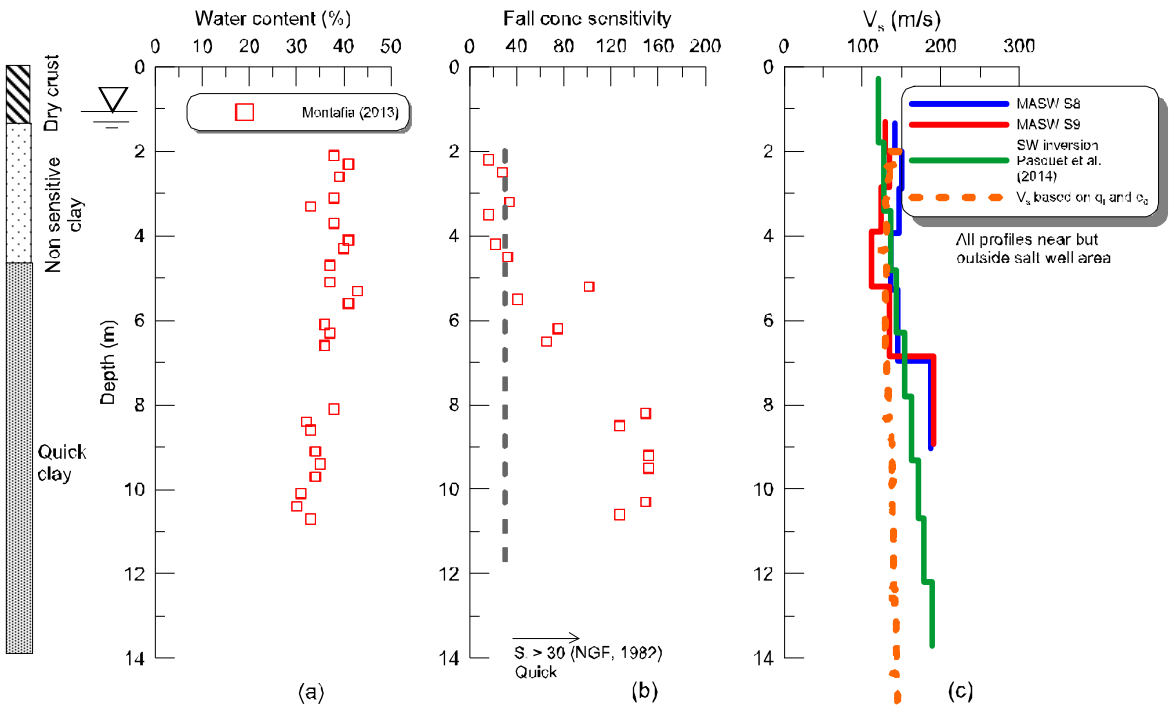


Figure B - 25: Dragvoll

B2.11 Rosten / Saupstad

The Rosten / Saupstad area is being investigated by NGI for the purposes on assessing the stability of potentially quick clay zones. The two MASW profiles at Rosten and Saupstad were obtained within about 200 m of one another. The soils have similar index properties, with the Saupstad clay being of slightly lower water content (Figure B-26). The Rosten soils are of medium to high sensitivity with no quick zones encountered. The Saupstad profile shows occasional quick zones.

Both MASW profiles show similar results (confirming earlier findings that MASW alone is not able to distinguish quick clay from other non-quick clays). The CPTU based method (using CPTU Rosten 9-3) based on q_t and B_q and on q_t and e_0 give reasonable predictions of V_s with the latter, as has been found earlier, not being able to capture the increase in V_s with depth. The formula based on q_{net}, σ_{v0}' and w underestimates V_s.

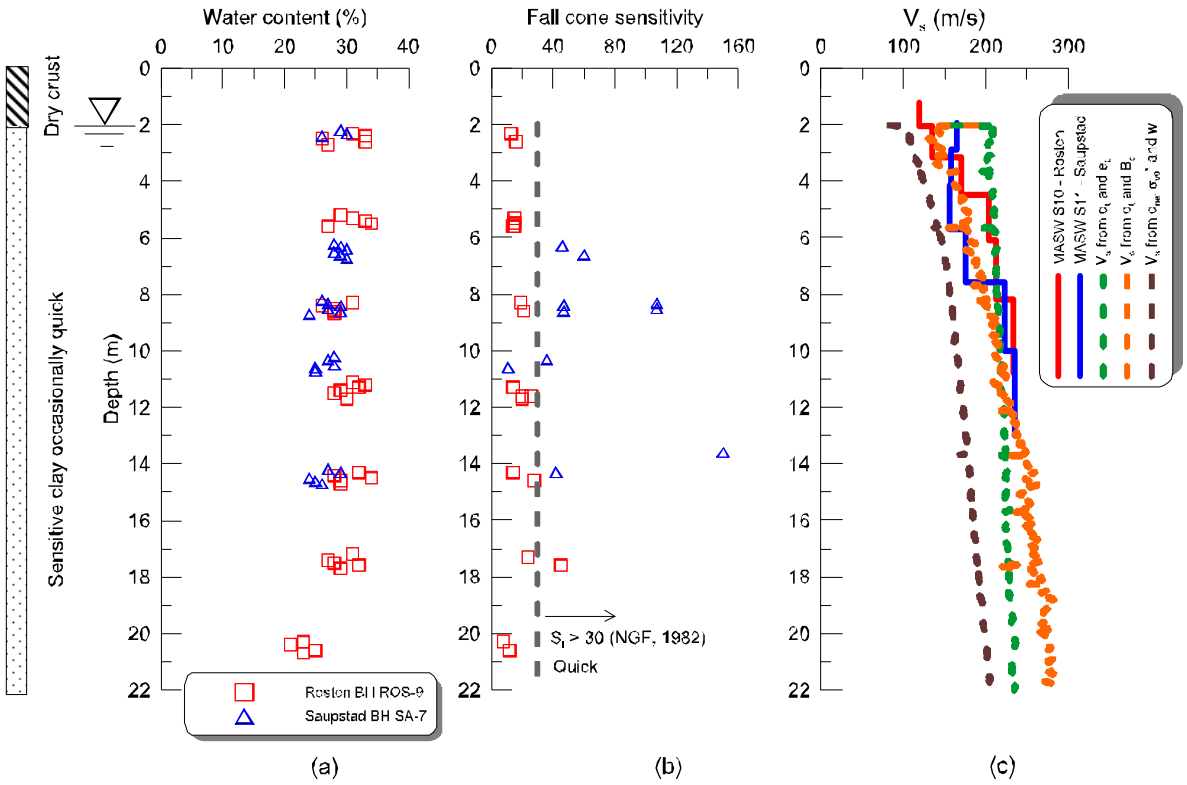


Figure B - 26: Rosten / Saupstad

B2.12 Rissa

The Rissa site is obviously well known due to the large quick clay landslide there in 1978. The testing here was carried out in an area just to the east of Rein Kirke and west of Lake Botn. Some results of index testing and MASW V_s measurements are shown on Figure B-27. The V_s values are relatively low but were generally consistent across the area studied.

All of the CPTU based techniques capture the measured V_s well. Arguably the techniques based on q_t and B_q and on q_{net}, σ_{v0}' and w work best and well capture the increase in V_s with depth.

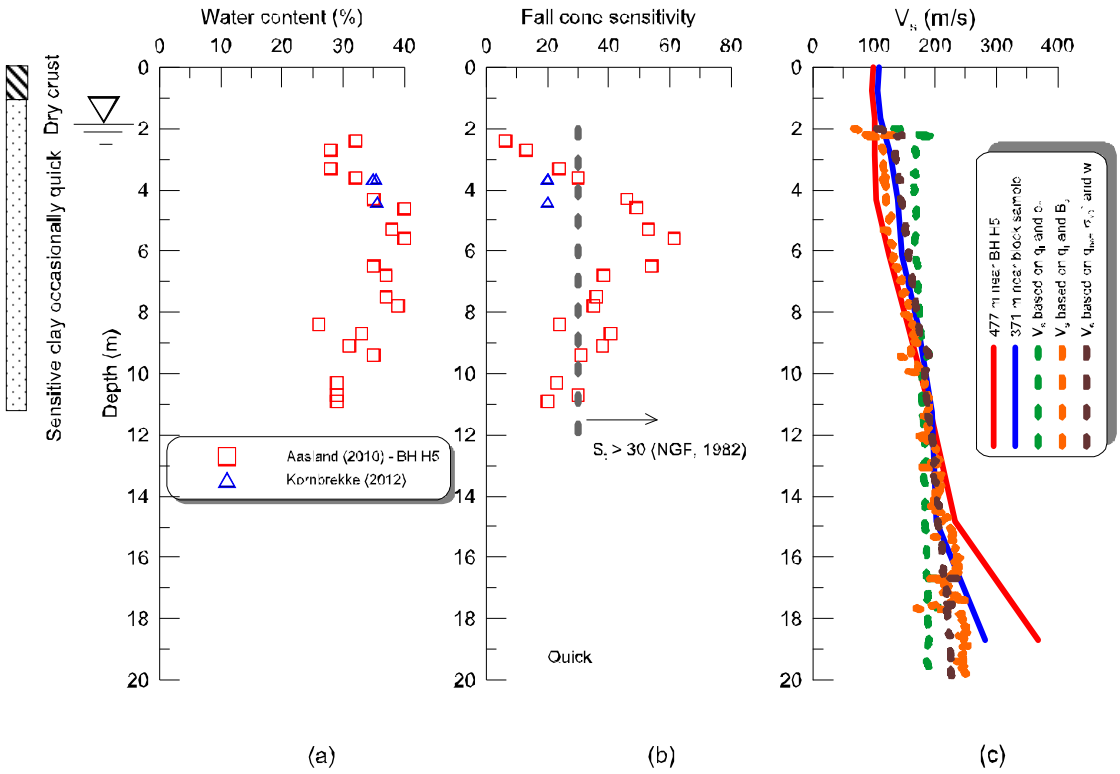


Figure B - 27: Rissa

B2.13 Hoseith / Okstad

Data is also available for two sites in the Trondheim area at Hoseith and Okstad where MASW testing was done by APEX as part of investigations at two commercial projects. So far no other soils data is available. Representative V_s profiles for the two sites are shown on Figure B-28. Values are similar for the two sites but seem higher than for other Trondheim data.

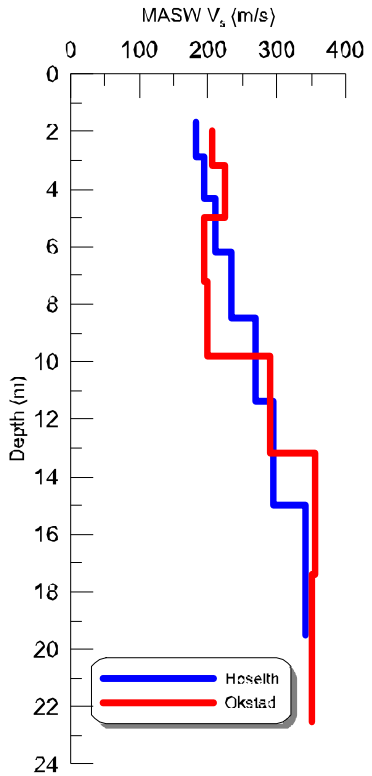


Figure B - 28: Hoseith / Okstad

B2.14 Kattmarka

The Kattmarka site near Namsos has been well studied due to the quick clay landslide which took place there in 2009, see (Nordal et al., 2009). The site is underlain by a thick deposit of layered clay, which is occasionally quick, see Figure B-29. V_s values range from about 100 m/s at ground level to about 180 m/s at 20 m. These values are at the lower end of those measured elsewhere in Norway, see Figures 9 and 10 of the main report.

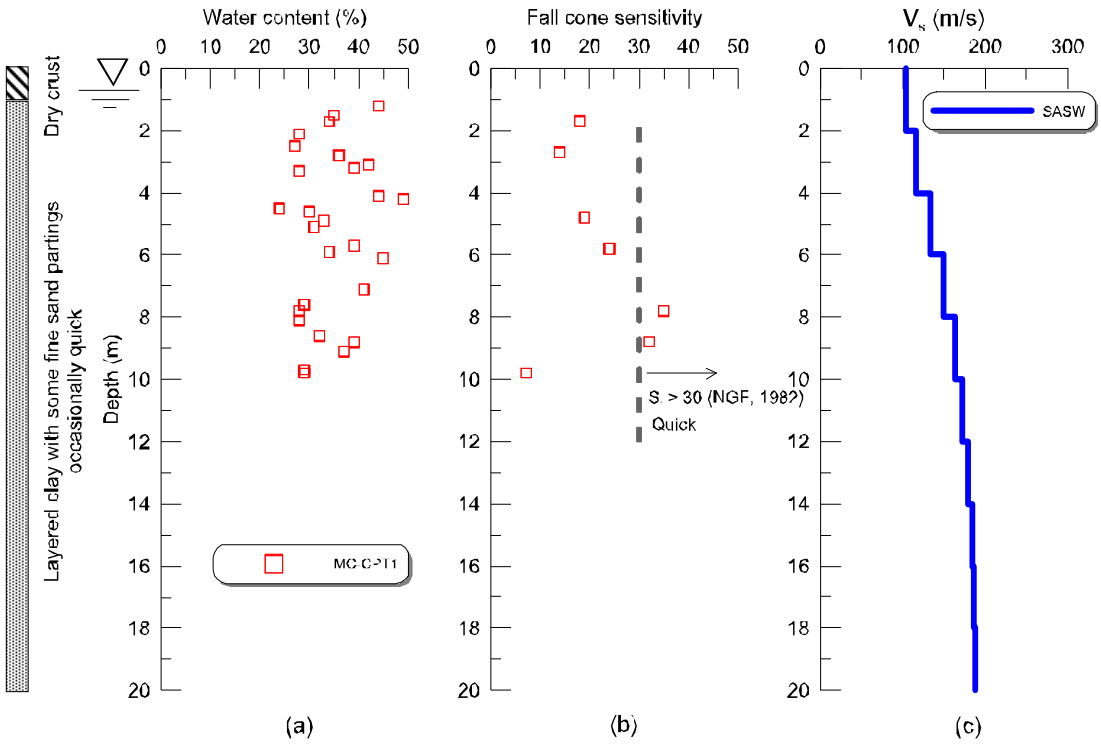


Figure B - 29: Kattmarka

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Kontroll- og referanseside/ Review and reference page

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