STATENS GEOTEKNISKA INSTITUT
SWEDISH GEOTECHNICAL INSTITUTE

Geophysics in soil mechanics

– in situ shear moduli determined by SASW-technique and more traditional geotechnical methods

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Varia 508

LINKÖPING 2001

Varia	Swedish Geotechnical Institute SE- 581 93 Linköping
Order	SGI Literature service Tel: 013–20 18 04 Fax: 013–20 19 09 E-mail: info@swedgeo.se Internet: www.swedgeo.se
ISSN	1100-6692
ISRN	SGI-VARIA--01/508--SE
Project number SGI	10177
Dnr SGI	1-9909-544
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2001-08-08

Preface

This report deals with the results from tests performed with a seismic surface method in a number of test fields, which have previously been thoroughly investigated with traditional geotechnical investigations and reported in several SGI-publications. The method is named SASW (Spectral Analysis of Surface Waves) and uses the spectral analysis of the surface waves to determine the maximal dynamic shear modulus.

In the report a comparison is performed between results from the SASW-method and results from seismic cone penetration tests and other geotechnical investigation methods.

The project is performed in cooperation between the Swedish Geotechnical Institute (SGI) and the Division of Soil Mechanics and Foundation Engineering and the Division of Engineering Geology at the Department of Geotechnology at Lund University of Technology. Worldwide and also in Sweden there is an ongoing discussion and ambition to implemented geophysical methods in soil mechanics design. This project is one contribution on that theme and hopefully there will be more in the future.

The authors wish to express their thanks to Carl-Axel Triumf and Rolf Larsson for many valuable points of view of the manuscript and the project. Special thanks to Sara Blomberg who has brought order in the confusion of the manuscript the authors left.

Malmö and Lund, August 2001

Björn Möller Mats Svensson

Contents

1 Background

In civil engineering, there is a great desire to implement geophysical methods in the geotechnical investigation process. Reasons fore this are an ambition to find a surface covering method which gives continuous information in between the geotechnical investigation points as well as finding a non penetrating testing method which is less space and resource demanding.

Various geotechnical parameters are today determined by site- and laboratory investigations. Site investigations require resources such as drilling rigs and mechanical equipment. They are normally penetrating in selected points. The investigations yield point information and no information about variations in between the different investigation points is given.

Development in geophysical methods has been great during the last decade, chiefly concerning surveying and evaluating. This has implied that the accuracy of the results is approaching the geotechnical range and the methods are now well suited for integration into the geotechnical investigation process.

In the last few years, internationally as well as in Sweden, a new method to determine the dynamic shear modulus (maximal shear modulus) and its variation in the soil mass has been developed. This method is designated as SASW (Spectral Analysis of Surface Waves).

This report accounts for surveys performed with the SASW-method in a number of test fields which have previously been thoroughly investigated with traditional geotechnical investigations by SGI (Swedish Geotechnical Institute). Also the shear moduli in these fields have been determined with seismic SCPT tests (Seismic Cone Penetration Test). These investigations have been accounted for in several SGI-publications.

2 Aim

The project is intended as a demonstration project in the fields of soil mechanics, geophysics and geodynamics. The aim of the project is to verify the usefulness of the SASW-method to determine the variation of the shear modulus and its variation in soil masses and to illustrate its potential to become a support and complement to traditional geotechnical site investigations. Even if profiles are discussed in chapter 3, this report has only evaluated performed point investigations for the SASW-methode.

The project is a link in a series of activities aiming at implementing geophysical methods in geotechnical engineering. Through correlating the results from the method in different soil conditions to the previously known geotechnical properties, a basis is created for judgement of how to supplement and possibly to a certain extent replace the traditional site investigations with a more rational method. A method which not only reduces the cost of the geotechnical investigations but also increases the amount and the quality of the information about the investigated soil mass.

The results from this project and the simplicity of the method indicate a future possibility to perform qualified geotechnical investigations in a more rational way and at the same time enhance the surface coverage and continuity of the results. This also reduces the risk of misinterpretation of the results from single investigation points.

The report shows comparisons between results from SASW measurements and previously obtained results from test sites with soft clays, silts and clay till. Correlations between statically and dynamically determined parameters are shown. The correlations imply among other tings, that the parameters are determined at different strain levels in the different types of tests.

These results further show the repeatability and some limitations of the SASW-method.

3 Methods for determination of low strain shear modulus

3.1 Seismic methods in general / Introduction / Historical background

Civil engineers and geologists have used seismic methods for characterising the subsurface since the beginning of the 20th century, but already at an early stage the focus was set on oil prospecting. Refraction seismics using the P-wave velocity for interpretation was the first seismic technique to be developed. The method has been described in numerous textbooks, for instance (Telford et al, 1990), discussing its benefits and drawbacks and will not be further discussed here. For civil engineering purposes, the use of refraction seismics has so far been limited to approximate determinations of horizontal strata, that is stratigraphical information. By the end of the 1920's, reflections were detected in the data from refraction surveys, and thereafter the reflection seismic technique was developed. Today, reflection seismics is the basic survey method in petroleum exploration. Until the mid 80's, reflection seismics had been used for civil engineering purposes only to the same limited extent as refraction seismics. One of the main reasons for this may be the problem in detecting the properties of the shallowest twenty meters, which are the levels most interesting for the civil engineers. However, for the last years great improvements have continuously been brought forward by the reflection seismic community, in the ability of handling also the shallowest depths. Since the end of the 90's, reflection seismics (in combination with surface wave seismics) is close to become a practically useful tool for determining true mechanical properties of the subsurface (Miller et al, 1999). Also the basic reflection seismic technique is described in a number of textbooks, (e.g. Telford et al, 1990), and will not be discussed further here.

From the seismic refraction and reflection techniques, the continuous vibration technique was developed in Germany in the 1930's (Hertwig, 1931). At that time it was not quite clear whether it was a shear wave or some other kind of wave that was recorded and used for interpretation. In the 1950's (e.g. Jones, 1958), it was clarified that the mechanical vibrating sources that were used to the greatest extent produced Rayleigh waves. The technique is today referred to as surface wave or Rayleigh wave seismics. At that time also the theory for how to interpret the collected data was known, but since only analytical solutions to the equations were available, only the simplest cases could be solved (Jones, 1958).

During the 1960's and 1970's a lot of effort was put into developing Cross-Hole , Down-Hole and Up-Hole seismic techniques in the civil engineering industry (Stokoe and Woods, 1972). With these techniques, shear waves are generated in predrilled boreholes or on the ground surface. Since the techniques use either sources or receivers on a known level in a borehole, it is clear how to relate the interpreted properties (shear moduli) to the true depth.

The method this report is focusing on, the Spectral Analysis of Surface Waves (SASW), and a second type of a seismic surface wave technique, the Continuos Surface Wave method (CSW) (Gordon 1997), have been developed simultaneously from the beginning of the 1980's. Both methods are developments of the continuous vibration technique mentioned above and became possible because of the computational power now available. The difference between the two methods is related to the energy source: the SASW technique uses an impact source (e.g. a sledgehammer) for generation of the surface waves whereas the CSW method uses a vibrational source. Both techniques make use of the dispersive character of a surface wave travelling through a layered media. The Rayleigh wave velocities are determined by spectral analysis of the surface waves that are generated. The SASW technique is described in detail in Chapter 3.2.1.

The next chapter in the development of seismic techniques was written in 1984 when the Seismic Cone Penetration Test (SCPT) was presented (Campanella et al, 1986). This method combines a seismic downhole test with a conventional CPT test and thus provides information on both stiffness and strength in one single sounding.

A new era of making use of seismic methods in civil engineering began by the end of the 1990's. The reflection seismic community, instead of treating the surface waves as ground roll, noise and nuisance, started to analyse the surface waves and use them for interpretation, thus making determination of the shear wave velocities of the most shallow depths possible. Recently published research (Miller et al, 1999; Xia et al, 2000) presents the Multichannel Analysis of Surface Waves method (MASW) producing high resolution 2-D profiles of shear wave velocities to depths of about 100 m using impulse sources like drop weights. The method is a combination of reflection seismics and SASW (Miller et al, 1999). Although these research results have not yet to any greater extent reached outside the geophysical society, there is a large potential in converting the shear wave velocities along a profile into the various moduli the geotechnical designers need.

3.2 SASW – Spectral Analysis of Surface Waves

In surface wave seismics, the shear modulus is determined by measurement of the Rayleigh wave velocity in situ and through that an evaluation of the shear wave velocity. The physical phenomenon making the SASW method possible is dispersion. Surface waves are generated in one point, propagating through the soil mass and measured in at least two other points on the ground surface, *Figure 1*. The velocity of the propagation depends on the frequency of the wave. The frequency variation of the velocity is called dispersion. High frequency signals (waves) are propagating near the ground surface whereas lower frequencies (long wavelengths) affect a larger volume and propagate through both the near surface layers and deeper down in the soil profile. The soil is assumed to be layered, which is one of the environments where dispersion occurs. By generating a wide range of frequencies, the SASW method uses the dispersion phenomenon to yield a continuous depth - velocity relation of a soil profile, see *Figure 2*.

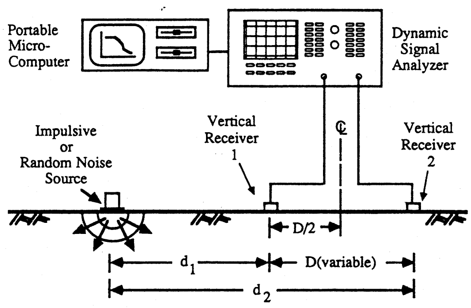


Figure 1. SASW field set up. (Rix et al 1991)

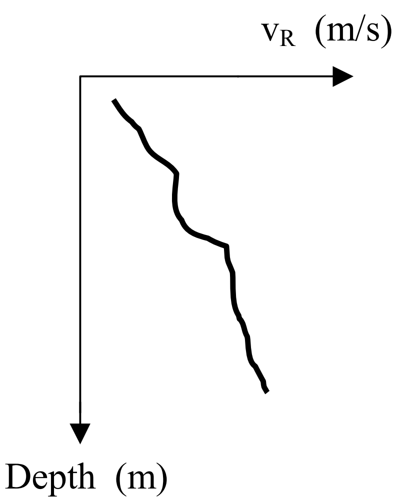


Figure 2. Dispersion curve.

The origin of the SASW-method is found in the Steady-State Rayleigh wave technique, developed in the 1950's and 1960's (Jones, 1958 and 1962, and Heukelom and Foster, 1960). When the Steady-State Rayleigh wave technique is used, a vertically acting vibrator producing a single frequency at a time is placed on top of the ground surface. A geophone is then successively moved away from the vibrator in order to find the distances where the vertical amplitude is in phase with the vibrator. The actual wavelength is found as the distance between two such maxima. Since the frequency of the vibrator is known, the Rayleigh wave velocity can be determined. The actual depth corresponding to the velocity is assumed to be half the wavelength. Performing a number of tests, varying the frequency of the vibrator, a full dispersion curve for the soil profile can eventually be interpreted. However, this procedure is time consuming.

The development of the modern SASW-method was presented in the beginning of the 1980's, see for instance Nazarian and Stokoe (1984).

System configuration

In Figure 1 the schematic set up of a SASW field system is shown. The active source is the generator of the surface waves and can be of different kinds;

- Vibrator
- Impulse source
- Random noise load
- Swept sine load

A minimum number of two receivers is located on the ground surface at certain distances from the source. To monitor the surface waves propagating through the soil geophones are used as receivers. For stiffer materials like pavements also accelerometers are used as receivers. The receiver signals are digitised and recorded by the recording equipment, which consists of a dynamic signal analyser, a seismograph or a computer. In 1987, Sanchez-Salinero et al, (1987) and Sheu, (1987) studied the source - geophone distance relation and concluded that $d_2 = 2d_1$ was the best configuration, see *Figure 1*. They also recommended a reduction of the recorded data to $\lambda/3 < (d_2-d_1) < 2\lambda$, where λ is the wavelength. With this restriction in mind, two different field set-ups were proposed to optimise the use of the generated waves and penetration depth; the Common receiver midpoint set-up, *Figure 3*, and the Common source set-up. In this study the Common receiver midpoint set-up was used with a maximum and minimum distance between the receivers of 16 m and 1 m respectively.

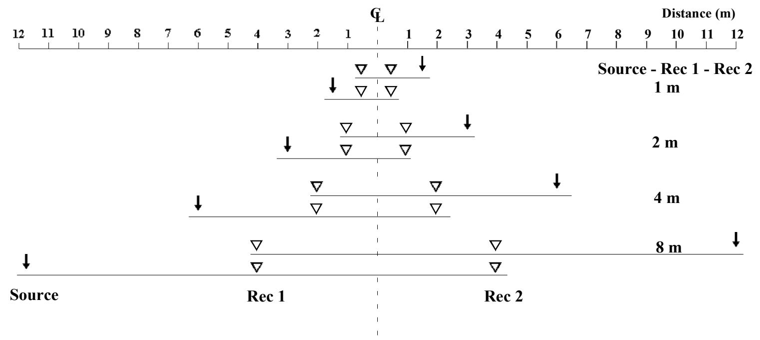


Figure 3. Common receiver midpoint set-up.

Analysis

An Fast Fourier Transform (FFT)-algorithm is used to transform the recorded signal from the time domain to the frequency domain. After calculation of a cross power spectrum the final result becomes a phase difference, $\Delta(f)$, between the two recorded signals for each frequency. A coherence function is used on site to check the current measurement and allows a direct on spot decision whether the test is acceptable or if it has to be repeated. An example of a coherence and the cross power spectrum plot on the screen of a Digital Spectrum Analyser can be seen in *Figure 4*.

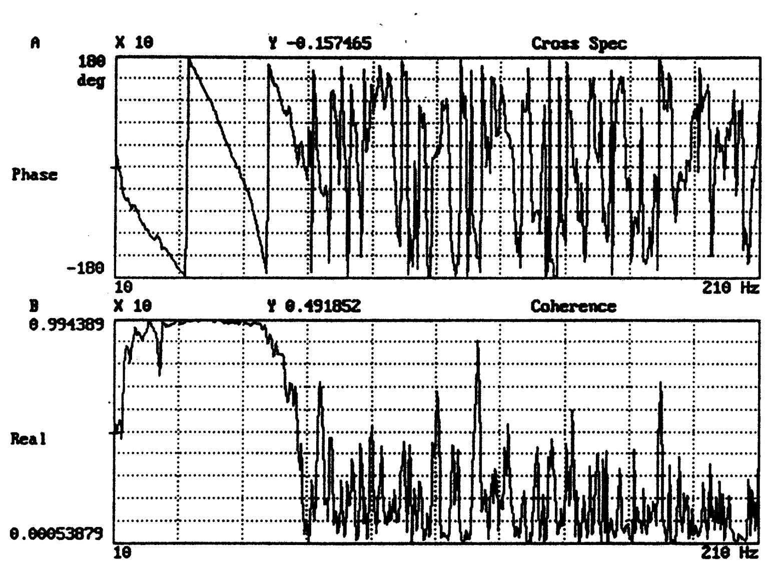


Figure 4. *SASW results plotted on site. Cross power spectrum from which the phase differences are determined (top) and coherence (bottom). A coherence > 0.95 is mostly regarded as acceptable.*

By applying Equations 1 - 3 below, first the time difference, *t*, between the two receivers, then the Rayleigh wave velocity, v_R, and finally the wavelength, λ, are obtained.

$$t(f) = \frac{\phi(f)}{2\pi f} \qquad \text{(Eq 1)}$$

$$v_R = \frac{d_2 - d_1}{t(f)} \qquad \text{(Eq 2)}$$

$$\lambda = \frac{v_R}{f} \qquad \text{(Eq 3)}$$

$$v_S \approx \frac{v_R}{0.92} \qquad \text{(Eq 4)}$$

$$G = \rho \cdot v_S^2 \qquad \text{(Eq 5)}$$

When the velocities have been calculated for all frequencies, a dispersion curve, v_R - λ or v_R – depth, can be plotted, Figure 2. The complete dispersion curve is built up by the results from all geophone distances, superposed as in *Figure 5*. Converting λ to depth *z* can empirically be made by assuming z = λ / 2 to z = λ / 3, (Heisey et al, 1982). However, an inversion method gives a theoretically more reliable depth determination and is recommended if the subsurface is of a heterogeneous character, (Roessetet al, 1991). The inversion technique is further described in chapter 3.2.1

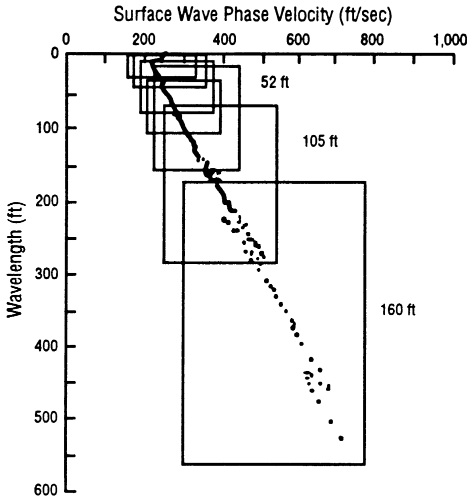


Figure 5. *Complete field dispersion curve - all geophone distances plotted together. (Rix, et al, 1991).*

According to Sanchez-Salinero et al, (1987), the ratio v_R / v_s ranges between 0.874 - 0.955 with Poisson's ratio, ν, ranging between 0 - 0.5. Assuming $v_R = 0.92v_s$ as in Equation 4 above, which is appropriate for an approximate interpretation, for example at a preliminary interpretation in the field (Sanchez-Salinero et al, 1987), the shear modulus is then obtained by using Equation 5.

3.2.1 Interpretation of SASW-data

The relation $z = \lambda /2$ or $z = \lambda /3$ (z = depth below ground surface, λ = wavelength for the Rayleigh wave) has for a long time been used for relating the velocities in the field dispersion curve to specific depths in the soil profile (Sanchez-Salinero et al, 1987). This is sometimes referred to as empirical inversion, and is still an acceptable procedure for an approximate interpretation. However, to make a theoretically more correct interpretation and to be able to handle more complex stratigraphies including, for example, low velocity layers, some kind of numerical modelling should be performed.

The transformation procedure can be stated as an **inverse** problem. A problem is called inverse when an effect of a physical system is observed but the properties causing this effect are unknown. Sometimes the term backward problem is used. The opposite is when the properties of the system are known but the effect they are causing is unknown. This is called the direct or the **forward** problem. The inverse problem that this method is dealing with is assigning depths to surface wave velocities and subsequently shear wave velocities and stiffnesses. On each site an effect is observed as a dispersion curve (phase velocity versus wavelength), but the properties of the soil - shear wave velocity, v_S, Poisson's ratio, ν, bulk density, ρ, and layer thicknesses, t, - that cause this effect are unknown.

The full inversion technique includes the forward modelling stage. An assumed soil model is given as input to the computer software, see *Figure 6*, and a theoretical dispersion curve is calculated and compared to the field dispersion curve. In the forward modelling case, the operator has to interact with the program and decide about changes in the assumed model if the fit between the two curves is not satisfactory. The inversion technique includes an algorithm that proposes a next model and also gives some figures of the quality of the model fit to the field data. This technique is more automated.

In this survey the forward modelling technique has been applied in a software named WinSASW (University of Texas, 1992) and the inversion technique in a software called SURF (Herrmann R, 1998). Parts of the forward modelling algorithms used in the programs will be presented below.

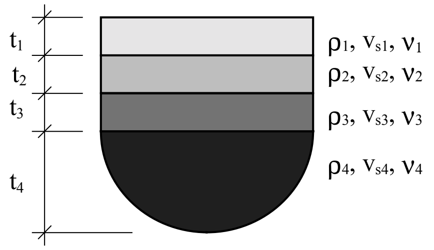


Figure 6. Assumed soil model. t - thickness of layer, ρ - density, v_s - shear wave velocity and ν - Poisson's ratio.

Definition of modelling problem to be solved

In the forward modelling technique for finding the most probable variation of the stiffness in a soil profile, the problem and aim is to find a solution to the wave equations below, Equations 6 - 7. By approximating the generated surface waves with the two dimensional solution for waves propagating along the surface of an elastic medium, the equations, as presented by Stokoe et al, (1994), to which a solution has to be found become;

$$\rho\frac{\partial^2 u}{\partial t^2} = M\frac{\partial^2 u}{\partial x^2} + (M-G)\frac{\partial^2 v}{\partial x \partial y} + G\frac{\partial^2 u}{\partial y^2} \qquad \text{(Eq 6)}$$

$$\rho\frac{\partial^2 v}{\partial t^2} = M\frac{\partial^2 v}{\partial y^2} + (M-G)\frac{\partial^2 u}{\partial x \partial y} + G\frac{\partial^2 v}{\partial x^2} \qquad \text{(Eq 7)}$$

u - displacements in the x (horizontal) direction
v - displacements in the y (vertical) direction
ρ - mass density of the soil
M - constrained modulus
G - shear modulus

The schematic picture in *Figure 7* describes how the numerical algorithm used in WinSASW for finding a solution to the Equations 6 and 7, i.e. the most probable variation of properties in the underground, is composed. The procedure starts with treating a single homogeneous layer and ends with a global stiffness matrix composed by a number of different layers, each including four different properties; t, ρ,.ν, v_S;

t - thickness of layer	ρ - density
ν - Poisson's ratio	v_S - shear wave velocity

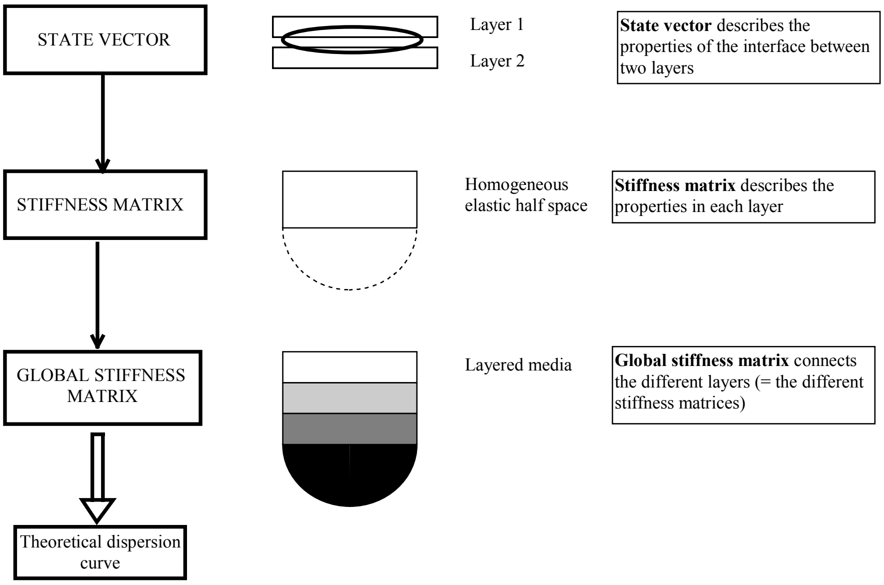


Figure 7. *Principle of the modelling algorithm.* ***1)*** *A state vector describing the properties of the interface between two layers is defined for each interface.* ***2)*** *Each state vector is related to the previous interface state vector by a transfer matrix defining the layer in between. By standard matrix algebra the transfer matrix is converted to a stiffness matrix allowing more efficient numerical solving algorithms to be used.* ***3)*** *All the stiffness matrices are combined in a global stiffness matrix composing the complete assumed soil profile.* ***4)*** *Finally the theoretical dispersion curve is calculated.*

More thorough descriptions can be found in Thompson (1950), Haskell (1953), Kausel and Roesset (1981) and Svensson (1998).

The inversion software SURF, (Herrmann 1998), only makes use of the Thompson-Haskell part of the algorithm presented above. It is used for the forward modelling part. The inversion algorithms used in SURF are not presented here.

3.3 Seismic cone

The seismic cone penetration test was originally developed at the University of British Columbia in Canada by Campanella and Robertson in the early 1980's . It is described in detail in Robertson et al, (1986) and Larsson and Mulabdic (1991). The equipment consists of an ordinary CPT or CPTU probe with a built-in velocity geophone (often measuring three components), a trigger on the ground surface and some kind of oscilloscope and data storage facility (normally both incorporated in a field or laptop computer), see *Figure 8*.

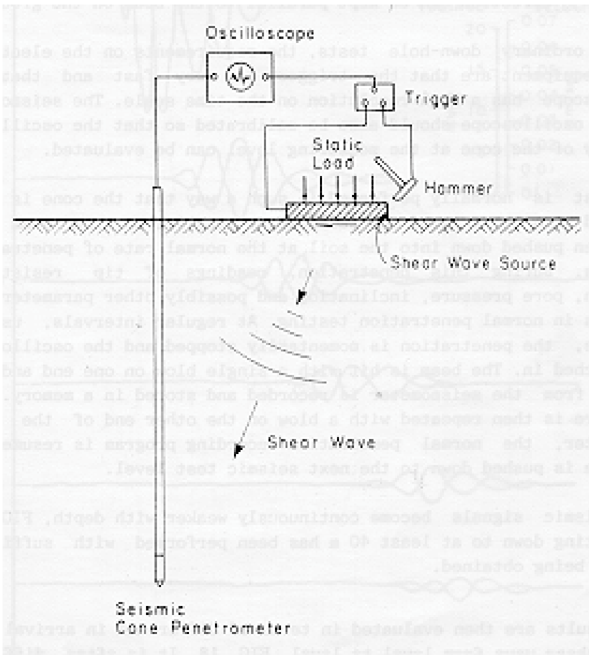


Figure 8. Schematic SCPT set up.

A sledgehammer hitting a loaded beam is normally used as the seismic source producing shear waves. When a SCPT test is carried out, the probe is first oriented with the axis of the geophone parallel to the beam axis and then pushed into the ground at the normal rate of penetration, 20 mm/s. The normal parameters; tip resistance, friction, pore pressure etc, are recorded continuously. At regular intervals, typically 1 m, the penetration is interrupted. The beam is hit with a single horizontal blow on one end and the geophone signal is recorded and stored. This procedure is then repeated with a blow on the other end of the beam. Thereafter the normal penetration continues.

The results are evaluated in terms of difference in arrival time of the shear wave from level to level, see *Figure 9*. The shear wave velocity in a certain soil layer is then calculated as $v_s = \Delta d/\Delta t$; where Δd = the travel distance for the wave between the test levels at the top and the bottom of the layer and Δt = difference in corrected arrival times at the top and the bottom of the layer (Larsson and Mulabdic, 1991).

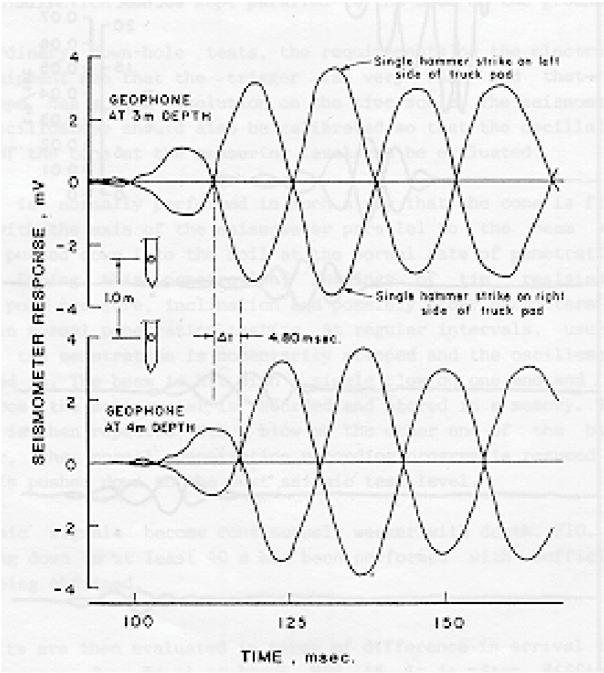


Figure 9. Interpretation of SCPT data.

3.4 Other methods

3.4.1 In situ tests

Other kinds of seismic methods often used for determination of the small strain modulus are Cross-Hole, Down-Hole and Up-Hole seismics. Detailed descriptions of those methods are found in for instance Stokoe and Hoar (1978). An optional method is very carefully performed plate loading tests, (Larsson and Mulabdic 1991).

3.4.2 Laboratory tests

The Resonant Column method (RC) has for a long time been used for measuring small strain stiffness in the laboratory, (Richart et al, 1970). During the 1980's, the bender element method was developed for the same purpose (Dyvik and Madshus, 1985). However, the bender element method is restricted to measurements of the initial shear modulus at very small strains whereas the strains can be varied within a certain interval in the RC method.

Bender element method

Compared to the RC tests, the bender element method is simpler and faster to carry out. The bender element device also allows repeated testing on the same specimen at different stages in various geotechnical tests, such as oedometer tests or triaxial tests. The bender element has been used in a wide range of materials (Souto et al, 1994).

By transmitting a shear wave with a piezoceramic element from the top of the soil specimen and recording it with another piezoceramic element at the bottom, see *Figure 10*, the travelling time for the shear wave through the specimen can be determined from the trace recorded by the oscilloscope, see *Figure 11*. The relation $G = \rho \cdot v_s^2$ is used to calculate G_{max}.

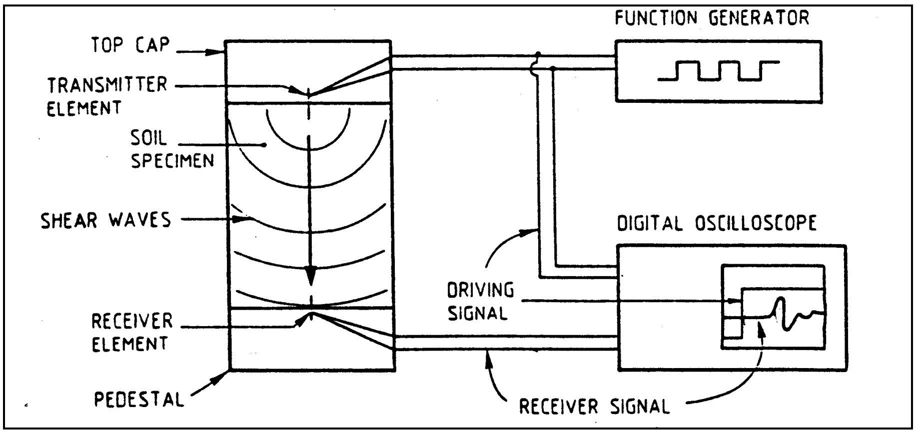


Figure 10. Bender element test set , (Dyvik and Madshus, 1985)

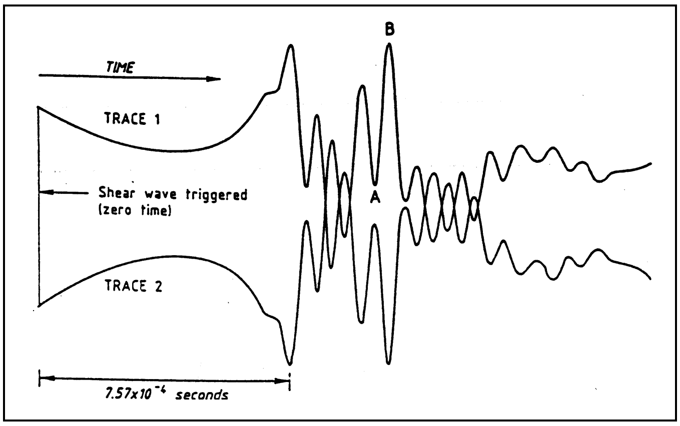


Figure 11. Trace from bender element test, recorded by a digitising oscilloscope. (Dyvik and Madshus, 1985)

4 Test sites

4.1 Norrköping

The test field at Norrköping is located north of the central parts of town, some 200 metres south of the Marieborg folk high-school and about 200 metres from the shore of Bråviken bay in the Baltic Sea.

The soil profile consists of about 14 metres of grey varved clay on top of friction material and rock. The dry crust is about 1 metre thick, but fissures and root threads extend to 2 metres depth. Thin layers or seams of silt occur at about 5 metres depth and from 7 metres depth thin silt layers occur regularly. These layers become thicker with depth.

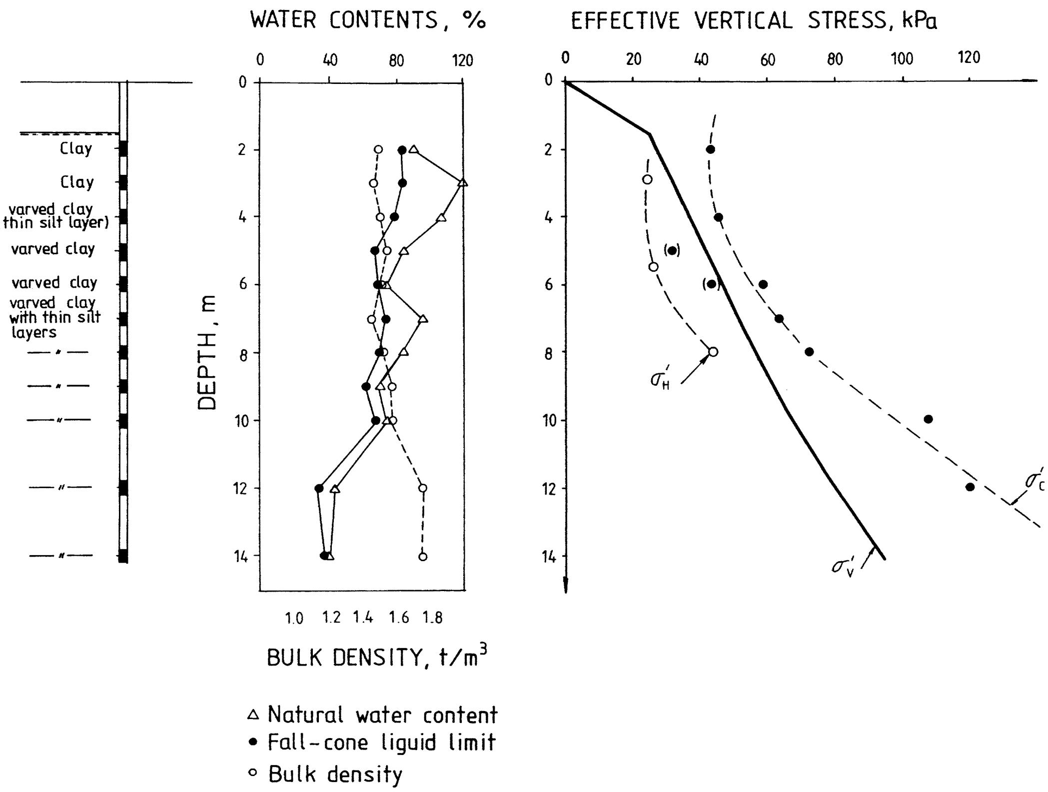


Figure 12. Soil profile at the test site in Norrköping, (Larsson and Mulabdic 1991).

The free ground water level is normally located 0.5 to 1.5 metres below the ground surface and the pore water pressure is hydrostatic from this level.

The natural water content is higher than the liquid limit and varies from 120 % at 3 – 4 metres depth to about 40 % in the silty bottom layers. The bulk density varies between 1.45 t/m^3 in the upper layers and 1.80 t/m^3 in the bottom layers with frequent thick silt layers. The overconsolidation ratio is estimated to have a minimum of about 1.2 between 4 and 6 metres depth. Above 4 metres depth, the overconsolidation ratio increases rapidly because of crust effects and below 6 metres depth it gradually increases to 1.5 – 1.6 at 10 metres depth. The undrained shear strength has a minimum of 10 kPa at 3 metres below ground surface and increases with depth to 16 kPa at 8 metres depth. It is almost constant between 8 and 12 metres depth. The sensitivity of the clay varies between 10 and 20.

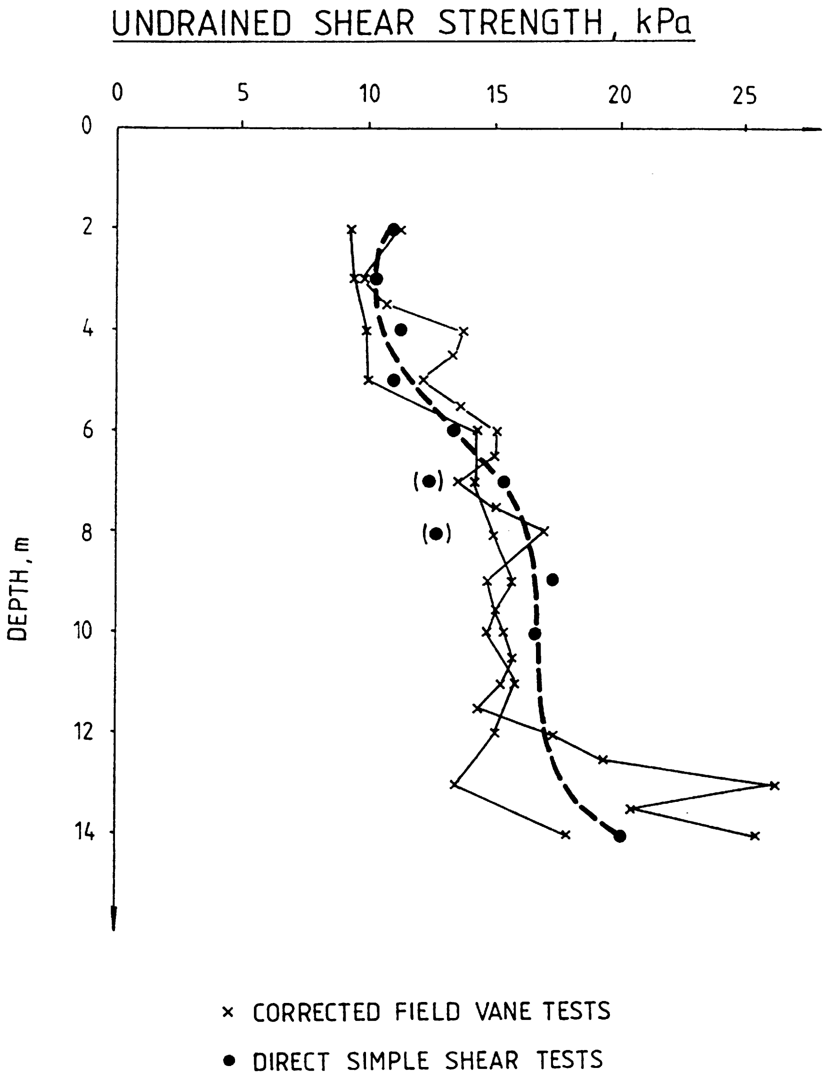


Figure 13. Undrained shear strength at the test site in Norrköping, (Larsson and Mulabdic 1991).

4.2 Lilla Mellösa

The test field at Lilla Mellösa is located north-east of Upplands-Väsby, about 40 kilometres north of Stockholm. The area is very flat.

The soil profile consists of 14 metres of clay on top of a thin sand layer and rock. At the top, there is a layer of organic top soil. The dry crust is unusually thin (0,5 metres) and consists of organic soil and lies on top of soft clay. The clay has an organic content of 5 % just under the crust, which decreases with depth and is less than 2 % from 6 – 7 metres depth. The colour changes from green to black and becomes grey with depth. The black colour results from sulphides. Below 10 metres depth, the clay becomes varved. The varves are at first diffuse, but become more and more pronounced with depth.

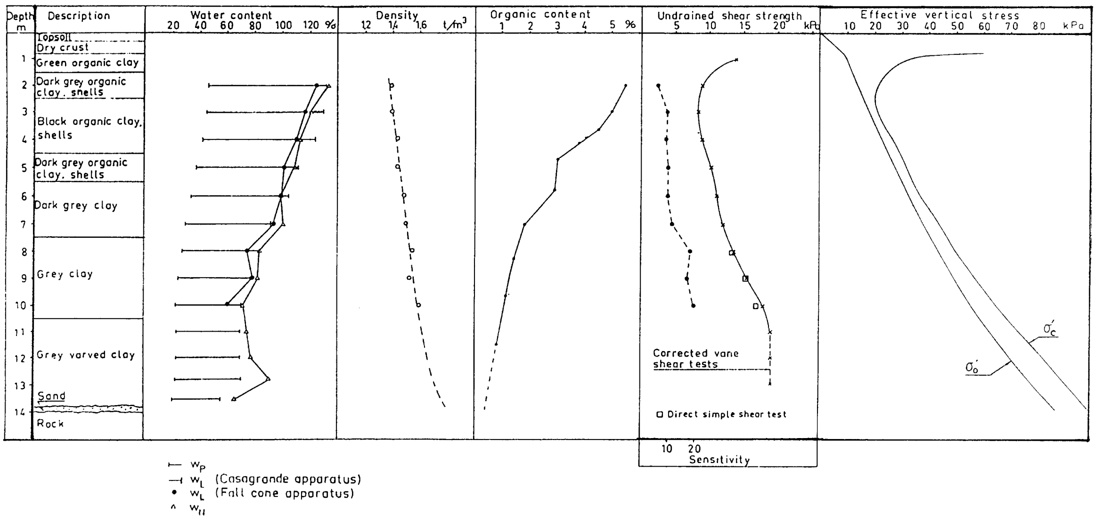


Figure 14. Soil profile at Lilla Mellösa, (Larsson and Mulabdic 1991).

The pore pressure in the ground has been found to be close to the hydrostatic pressure for a ground water level 0.8 metres below the ground surface.

The natural water content is approximately equal to the liquid limit and decreases from a maximum of about 130 % to about 70 % in the bottom layers. The bulk density increases from 1.3 t/m^3 to about 1.8 t/m^3 at the bottom. The undrained shear strength has a minimum of 8 kPa at 3 metres below the ground surface and then increases with depth. The sensitivity in the soil varies between 10 and 20. Overconsolidation occurs in the dry crust down to about 2,5 metres depth. Below this depth, the overconsolidation ratio is almost constant 1.2.

4.3 Vägverket (Borlänge)

The test field is located about 70 to 80 metres south-west of the buildings of the head office of the Swedish National Road Administration (Vägverket) in the city of Borlänge.

The soil profile consists of loose to very loose silty soils with a dry crust which is approximately 1.5 metres thick. The crust and the underlying soil consists mainly of medium silt. Between 4 and 5 metres depth, there is a layer of silty clay, followed by medium silt down to about 9 meters depth, where more clayey layers are found. Silt and layers of silty clay then alternate down to about 15 metres depth where coarser silt/fine sand is found. Below 20 metres depth, there is coarser sand, which is estimated to reach down to at least 40 metres below the ground surface.

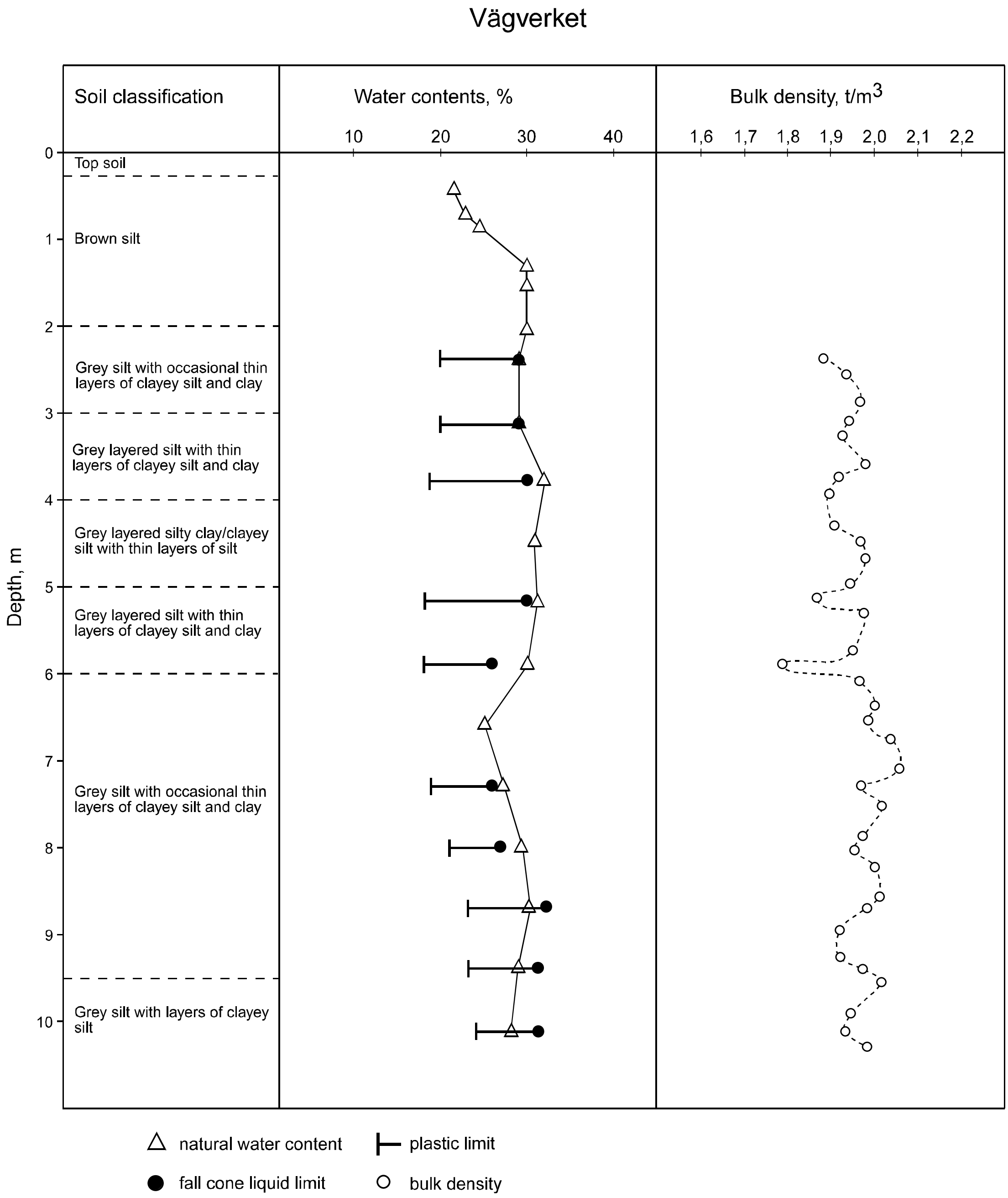


Figure 15. Soil profile in the test field at Vägverket (Borlänge), (Larsson 1997).

The ground water is artesian, with a water pressure in the coarser soil below 15 metres depth roughly corresponding to a hydrostatic head at the ground surface. The free ground water level in the upper soil layers varies between 1 and 2.5 metres below the ground surface.

The natural water content is about 30 % and the liquid limit is close to the natural water content. The bulk density of the soil varies mainly between 1.9 and 2.0 t/m^3.

4.4 Vatthammar

The test field at Vatthammar is located near Stora Tuna, about 5 kilometres south-east of Borlänge. It is located in a flat farmland area. About 150 metres north-west of the field, the ground slopes down into a deep ravine created by a small river, Tunaån.

The soil profile consists of mainly medium dense silt to great depths. The upper 6 metres consists of brown- grey layered silt with occasional thin layers of clayey silt or silty clay. Between 5 and 6 metres depth, the frequency of these clayey layers increase. Below 6 meters depth, there is a more uniform grey silt with only a few occasional thin clayey layers at about 8 metres depth.

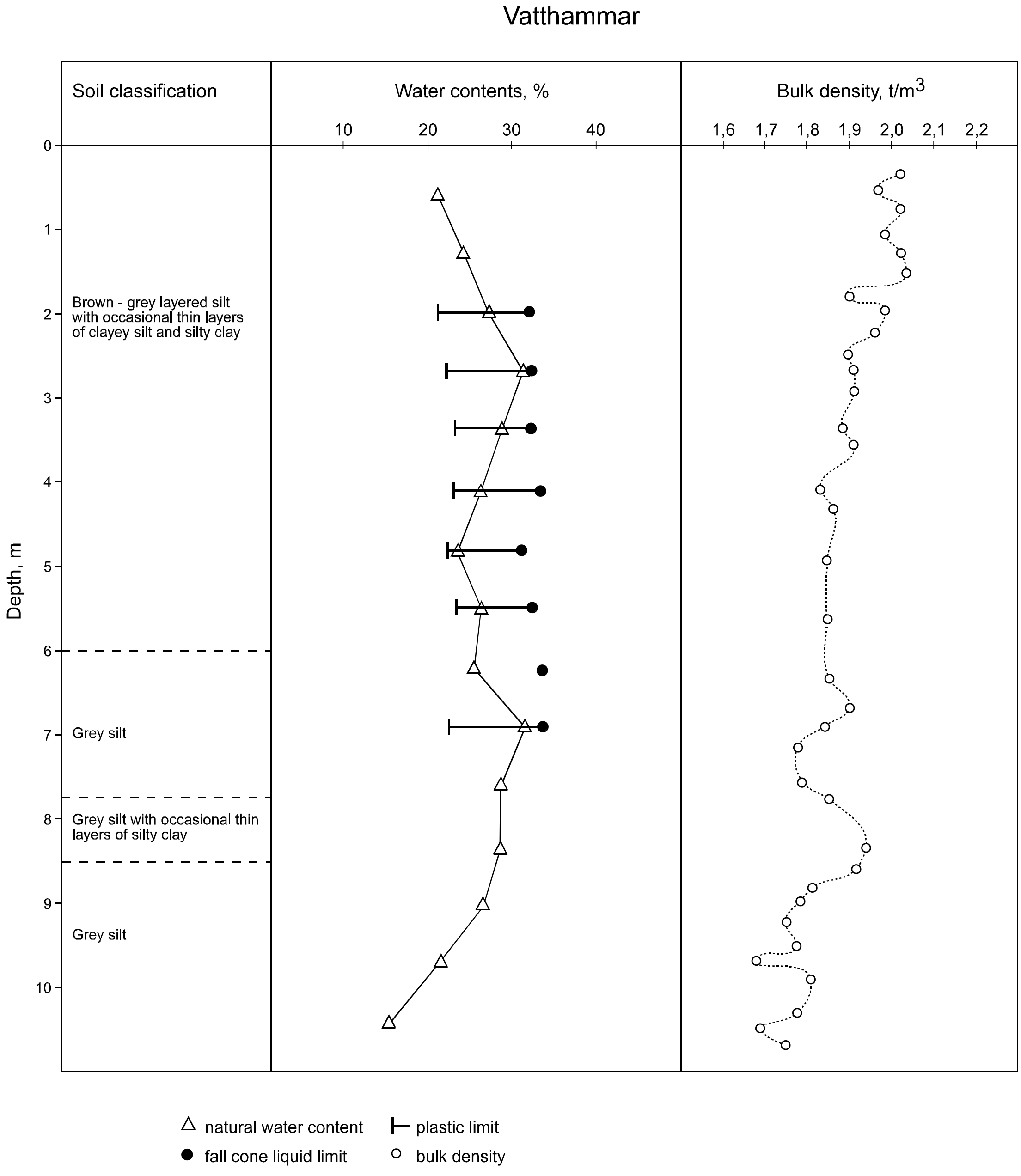


Figure 16. Soil profile in the test field at Vatthammar, (Larsson 1997).

Negative pore pressures were recorded down to a depth of about 18 metres below the ground surface. From this level, the recorded pore pressures corresponded to a hydrostatic pressure increase with depth. Also the measured negative pore pressures between 13.5 and 18 metres depth corresponded to the same hydrostatic pressure line, indicating that this zone was fully saturated.

The natural water content was about 26 % and the liquid limit about 32 %.

4.5 Tornhill

The test field is located north of the city of Lund, about 1 kilometre outside the limits of the built up areas in the city. It is a former market garden surrounded by cultivated farmland. The area is very flat and consists of open grassland.

The soil profile at the test site consists of about 3 metres of Baltic clay till, followed by 3 metres of an intermediate layer consisting of a mixture of North-east clay till, sedimentary deposits of sand and silt and Baltic clay till. 8 meters of North-east clay till is found beneath and then bedrock of clay shale. Accordingly, the depth to the bedrock is about 14 metres. The North-east clay till is dominated by material originating from crystalline rock and the Baltic clay till by material originating from sedimentary rocks.

The free ground water level in the upper soil layers in the test field varies seasonally between the ground level and approximately two metres below. In the upper two main layers and in the upper parts of the North-east till, the pore water pressure increases more or less hydrostatically from this free ground water level. At greater depths, the pressure drops towards a new and considerably lower free ground water surface in the bedrock.

The natural water content decreases with depth from between 13 and 23 % in the upper 3 metres down to about 10 % at 8 metres depth. Below this level it appears to be fairly constant. The liquid limit varies between 20 and 45 % in the upper layers and decreases down to about 20 % at 8 metres depth. The average bulk density is about 2.15 t/m^3 down to a depth of 6 metres whereupon it increases to become about 2,4 t/m^3 from 7 metres depth and downwards. The soil in all the profile is overconsolidated or heavily overconsolidated. The field vane tests showed an undrained shear strength of generally 300 to 400 kPa between 1,5 to 3 metres depth dropping to around 200 kPa between 3 and 5 metres depth and then increasing again.

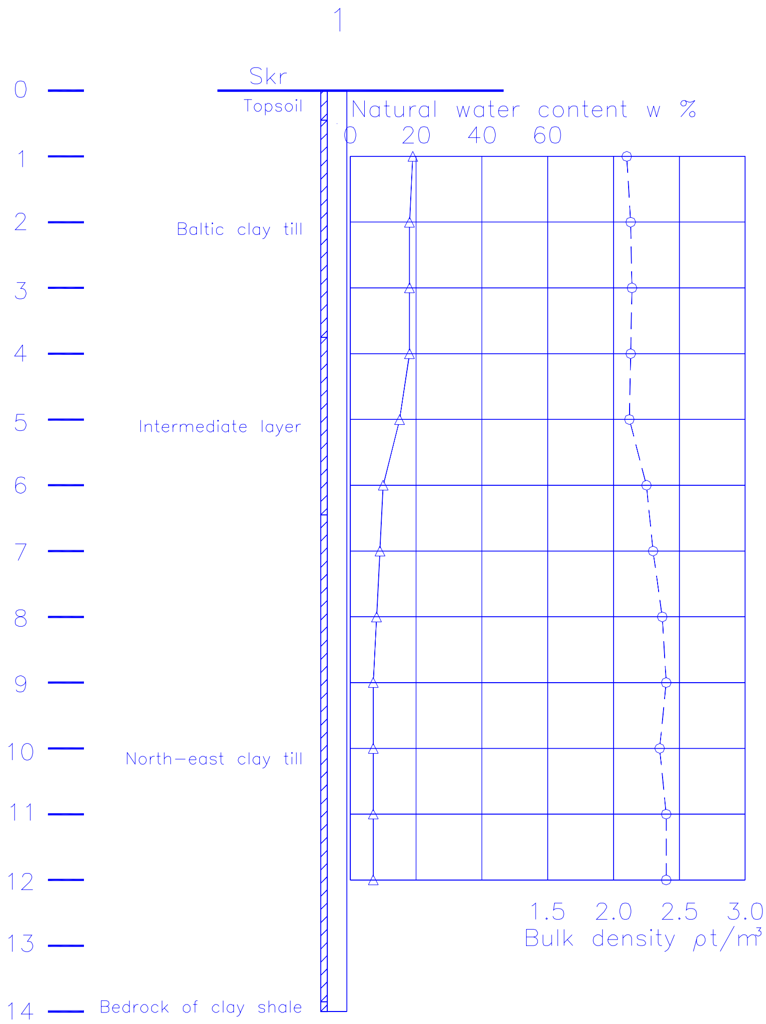


Figure 17. Soil profile in the test field at Tornhill, (Larsson 2000 and Dueck 1994).

5 Results

General

The unprocessed field data for each geophone distance consists of two files with frequency and corresponding phase difference in the first file, and frequency and corresponding coherence in the second file. By using the software WinSASW, data reduction is carried out based on the coherence data, and the dispersion curve for each line is constructed. In this project interpretation has then been performed in mainly two ways; Empirically and Inversion with the software SURF (Herrmann R, 1998). However, for the interpretation of the data from the Mellösa embankment only forward modelling instead of full inversion had to be used because of numerical reasons. The numerical problems were caused by the stiffer layer on top of a less stiff layer. Inversion technique is otherwise generally regarded as more robust and reliable than forward modelling, provided the inversion algorithm is proper enough. Most often there is a better control of the quality of the solution.

The empirical interpretation was made using $G = \rho\,(v_R/0.9)^2$ and $z = \lambda/2$. In the inversion interpretation, the SGI seismic cone velocities (SCPT) and layer thicknesses were given as a start model to the SURF program. In general it can be stated that all available a priori information shall be used for establishment of the start model. If no other information is available the empirical interpretation should be used. Any information of the layering from other methods will improve the possibilities for determination of the stiffness profile.

The SASW data are compared to data from the seismic cone tests (SCPT) and to the ordinary CPT data in terms of the uncorrected tip resistance q_c. An empirical relation proposed by Mayne and Rix, (1993) was used to estimate G_0 from the q_c data, see Equation 8.

$G_0(q_c) = (99.5 p_a^{0.305} \times q_c^{0.695}) / (e_0^{1.13})$; p_a = 100 kPa, e_0 = porosity (Eq 8)

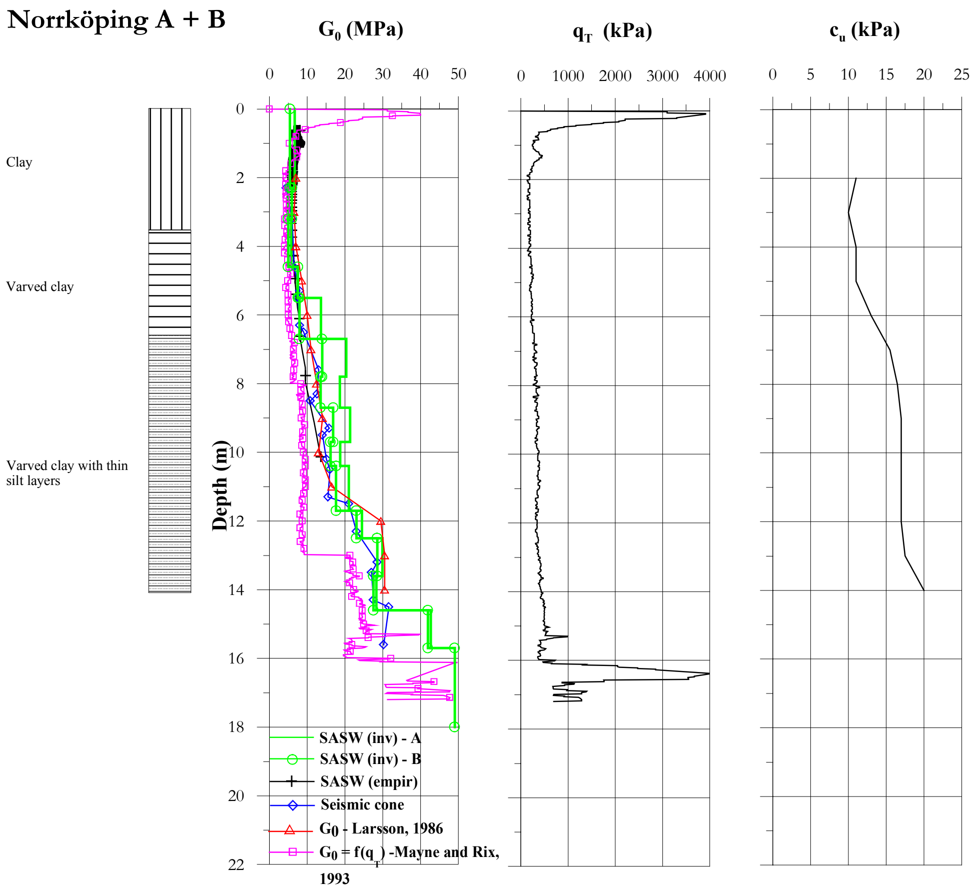
e_0 was interpreted from the SGI-reports under the assumption of full saturation.

A relation proposed by Larsson and Mulabdic (1991),

$G_0 = ((208/I_p + 250)c_u)$, (Eq 9)

was also used for comparison with the SASW results in the clay profile at Lilla Mellösa. For the other clay profile at Norrköping data plotted in Larsson and Mulabdic (1991) after a similar relation proposed by Larsson (1986) was used.

5.1 Norrköping



A

Depth (m)	vs (m/s)	Poisson's ratio	Density (t/m3)	Gmax (MPa)
2,3	66,50	0,35	1,5	7
3,2	58,00	0,35	1,5	5
4,6	62,00	0,35	1,5	6
5,5	70,40	0,35	1,5	7
6,7	95,20	0,35	1,5	14
7,8	112,70	0,35	1,6	20
8,7	107,80	0,35	1,6	19
9,7	115,50	0,35	1,6	21
10,4	108,00	0,35	1,6	19
11,7	114,60	0,35	1,6	21
12,5	116,70	0,35	1,8	25
13,6	128,80	0,35	1,8	30
14,6	125,40	0,35	1,8	28
15,7	130,70	0,35	2,5	43
18	141,50	0,35	2,5	50

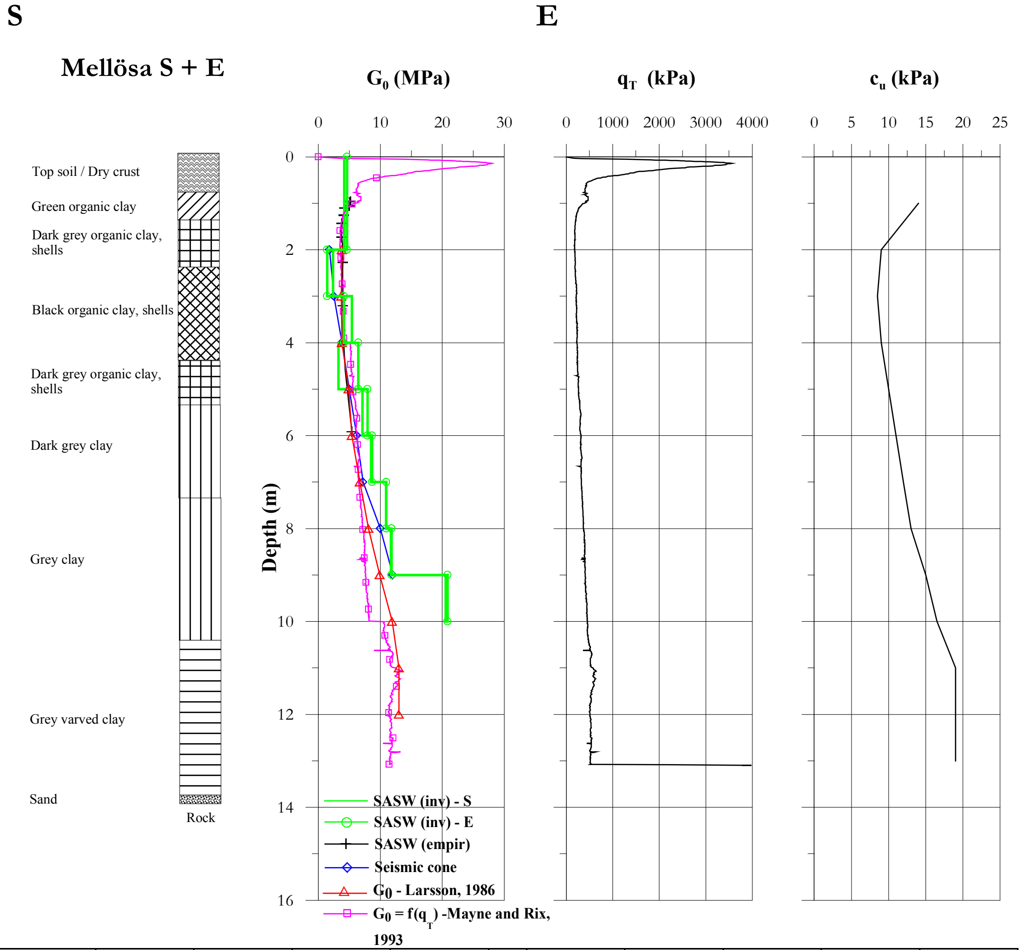
B

Depth (m)	vs (m/s)	Poisson's ratio	Density (t/m3)	Gmax (MPa)
2,3	60	0,35	1,5	5
3,2	63	0,35	1,5	6
4,6	57	0,35	1,5	5
5,5	71	0,35	1,5	8
6,7	73	0,35	1,5	8
7,8	92	0,35	1,6	14
8,7	103	0,35	1,6	17
9,7	100	0,35	1,6	16
10,4	105	0,35	1,6	18
11,7	113	0,35	1,8	23
12,5	126	0,35	1,8	28
14,6	124	0,35	1,8	27
15,7	130	0,35	2,5	42
18	140	0,35	2,5	49

Figure 18. Results and final models at Norrköping. For better resolution see Appendix A..

In general there is a reasonable agreement between the results from the two parallel SASW-lines except for a zone between 6-10 m where the A-line shows higher G_0-moduli, *Figure 18*. On the 6 m level there is also a clear increase of the shear strength. These changes correspond to the change in soil type to varved clay with silty layers. In the rest of the two soundings, the agreement between the SASW and the SCPT moduli is good. In the uppermost five meters of "homogeneous clay" both SCPT tests and the SASW measurements yield smooth moduli curves but when the silty layers occur, a larger G_0-variation between different depths can be seen. The empirical interpretation of the SASW data results in a smoother curve but the values of the moduli are similar to the SCPT values. Apart from the uppermost metre, the $G_0(q_c)$ is found to be a little lower than the other methods.

5.2 Lilla Mellösa



Depth (m)	vs (m/s)	Poisson's ratio	Density (t/m3)	Gmax (MPa)		Depth (m)	vs (m/s)	Poisson's ratio	Density (t/m3)	Gmax (MPa)
2	55	0,35	1,4	4		2	57	0,35	1,4	5
3	41	0,35	1,4	2		3	32	0,35	1,4	1
4	62	0,35	1,4	5		4	54	0,35	1,4	4
5	48	0,35	1,4	3		5	68	0,35	1,4	6
6	71	0,35	1,4	7		6	75	0,35	1,4	8
7	78	0,35	1,4	9		7	79	0,35	1,4	9
8	88	0,35	1,4	11		8	89	0,35	1,4	11
9	92	0,35	1,4	12		9	92	0,35	1,4	12
10	121	0,35	1,4	21		10	122	0,35	1,4	21

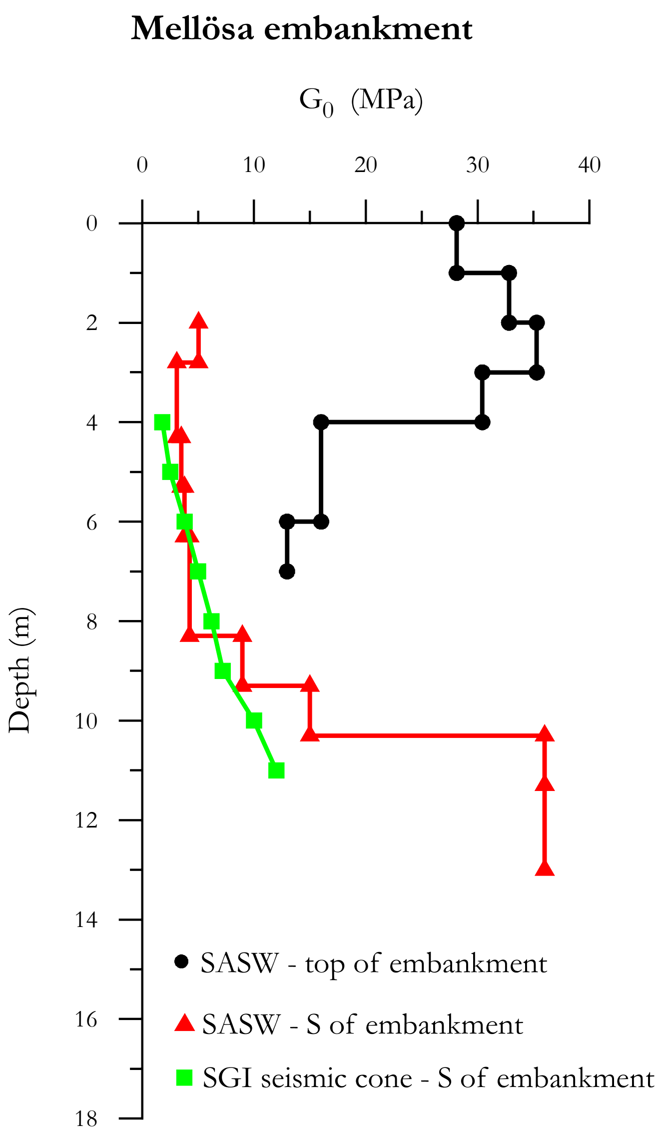
Figure 19. Results and final model at Mellösa. For better resolution see Appendix B.

Close to the ground surface the SASW results indicate a stiffer material, which is probably caused by the dry crust, see *Figure 19*. This is also indicated in the $G_0(q_c)$ data. Below this, there is a low stiffness layer interpreted between 2-3 m, corresponding to the dark grey organic clay layer found in the same zone. Another low stiffness layer is seen in the southern SASW-S line between 4-5 m. Also here, the dark grey organic clay may be the reason. From 4-5 m depth, the moduli are increasing with depth. This behaviour is also seen in the curves estimated from q_c and $c_u$values. When comparing the SASW and SCPT moduli, the SASW results yield slightly higher values. Also on this site the empirically interpreted SASW modulus curve shows a smoother trend and is in reasonable agreement with the SCPT moduli. Above 6-7 m depth, the $G_0(q_c)$ data are in agreement with SASW and SCPT data, but below the $G_0(q_c)$ values are lower.

Lilla Mellösa Embankment

On top of a 2.5 m thick test embankment of gravel, which was built already in the 1940's, an SASW test was carried out. The original stratigraphy of the soil below the embankment was identical to that presented for the Lilla Mellösa site. Since then, the soil has consolidated considerably for the load and the settlements in the central parts of the test embankment amounts to about 2.0 m. The moduli interpreted from the tests on the embankment were compared to the results from tests carried out south of the embankment. It is clearly observed that higher moduli are obtained in the upper gravely material, see *Figure 20*. As could be expected, the effect of long term consolidation below the embankment gravel is also clearly indicated in terms of increased moduli in the subsoil. Unfortunately the limited size of the embankment did not allow enough distance between the geophones in order to reach the desired full penetration depth.

The modelling of the Embankment data was carried out using the WinSASW software because the program SURF could not handle the stiffer uppermost layer properly.

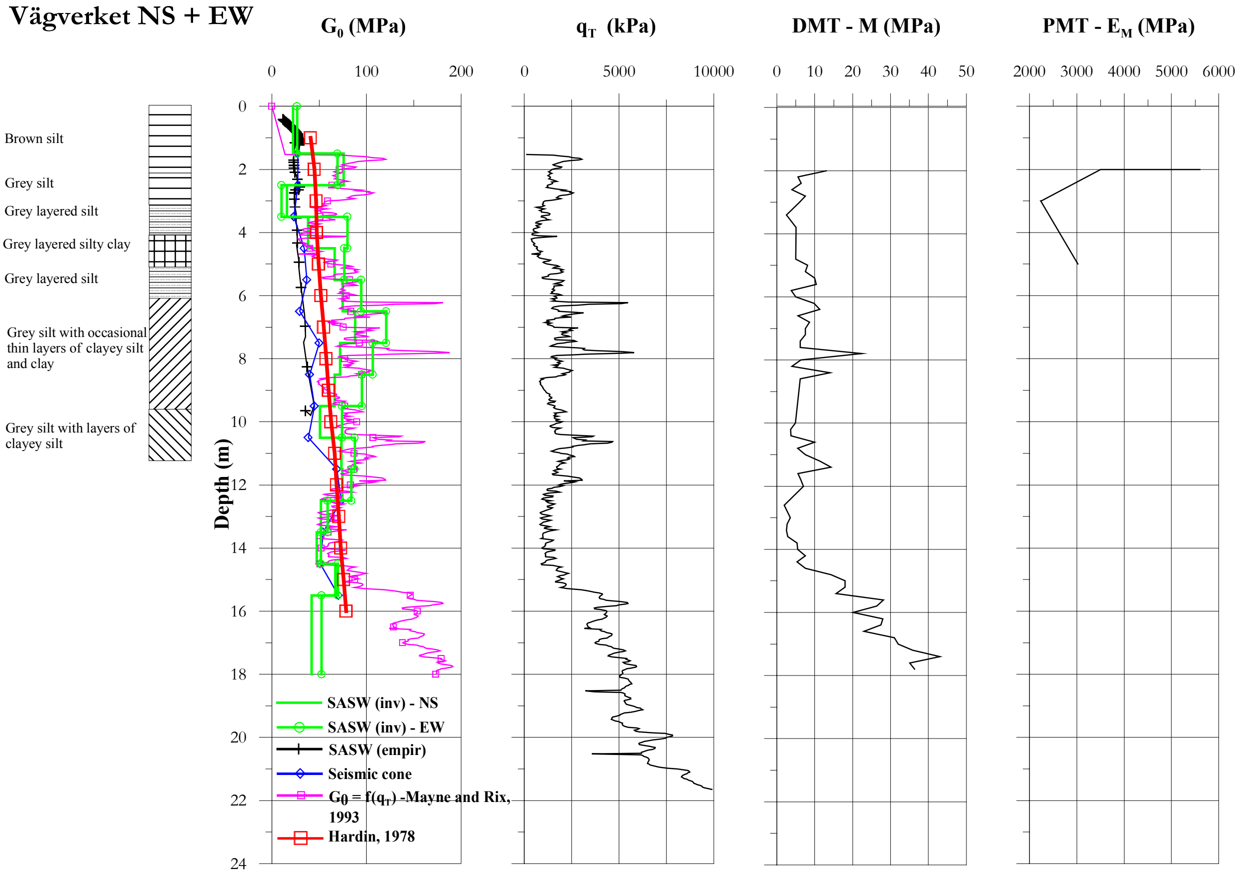


Embankment

Depth (m)	vs (m/s)	Poisson's ratio	Density (t/m3)	Gmax (MPa)
1	125	0,35	1,8	28,1
2	135	0,35	1,8	32,8
3	140	0,35	1,8	35,3
4	130	0,35	1,8	30,4
6	100	0,35	1,6	16
7	90	0,35	1,6	13,0

Figure 20. Results and final model for the embankment at Lilla Mellösa.

5.3 Vägverket (Borlänge)



NS

Depth (m)	vs (m/s)	Poisson's ratio	Density (t/m3)	Gmax (MPa)
1,5	107,3	0,35	1,95	22
2,5	197,1	0,35	1,95	76
3,5	90,8	0,35	1,95	16
4,5	140,2	0,35	1,95	38
5,5	184,6	0,35	1,95	66
6,5	195,8	0,35	1,95	75
8,5	192,3	0,35	1,95	72
9,5	184,4	0,35	1,95	66
10,5	161,5	0,35	1,95	51
11,5	194,1	0,35	1,95	73
12,5	195,3	0,35	1,95	74
13,5	163,3	0,35	1,95	52
14,5	155,8	0,35	1,95	47
15,5	185,6	0,35	1,95	67
18	146,9	0,35	1,95	42

EW

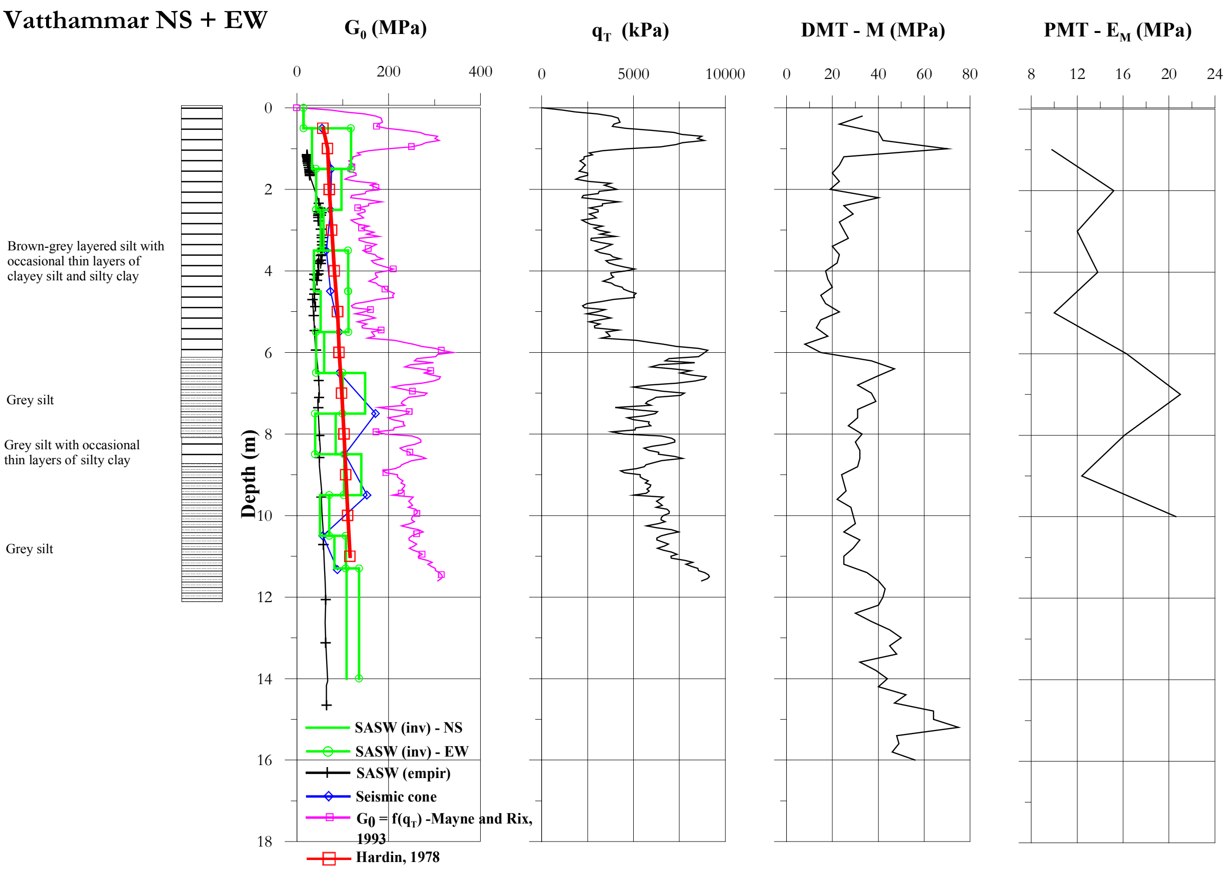
Depth (m)	vs (m/s)	Poisson's ratio	Density (t/m3)	Gmax (MPa)
1,5	117	0,35	1,95	27
2,5	188,7	0,35	1,95	69
3,5	72,7	0,35	1,95	10
4,5	202,4	0,35	1,95	80
5,5	198,2	0,35	1,95	77
6,5	220	0,35	1,95	94
7,5	248,9	0,35	1,95	121
8,5	233,9	0,35	1,95	107
9,5	220,9	0,35	1,95	95
10,5	195,2	0,35	1,95	74
11,5	211,9	0,35	1,95	88
12,5	207,7	0,35	1,95	84
13,5	174	0,35	1,95	59
14,5	163,4	0,35	1,95	52
15,5	189,2	0,35	1,95	70
18	164,4	0,35	1,95	53

Figure 21. Results and final model at Vägverket. For better resolution see Appendix C.

The G_0-curves are more irregular and the values change more rapidly with depth than for the clay sites presented above. In general, the interpreted SASW moduli are twice as high as the SCPT moduli in the upper 10 m, see *Figure 21*. Although the rapid changes of the SASW moduli roughly corresponds to the layering of the soil in the top 4 m, the large difference in moduli within these short distances is not realistic.

However, when comparing the shape of the SASW moduli curve with the q_T-curve there is a certain resemblance. Below 5 m depth, the moduli variations are less dramatic. This may be an effect of a more homogeneous silt between 6-10 m. The highest moduli found in the profile are between 7-9 m, which also corresponds to high q_T-values. The behaviour is similar for both SASW lines. The SCPT moduli also vary, but the changes in SASW moduli are more dramatic. However, there is a remarkable correlation with the empirical $G_0(q_c)$-interpretation. Again the empirical way of interpreting the SASW data results in a smoother curve following the SCPT trend quite well.

5.4 Vatthammar



NS

Depth (m)	vs (m/s)	Poisson's ratio	Density (t/m3)	Gmax (MPa)
0,5	83	0,35	2	14
1,5	129	0,35	2	33
2,5	220	0,35	2	97
3,5	170	0,35	2	58
4,5	136	0,35	2	37
5,5	161	0,35	2	52
6,5	180	0,35	1,85	60
7,5	283	0,35	1,85	148
8,5	214	0,35	1,85	85
9,5	276	0,35	1,85	141
10,5	167	0,35	1,8	50
11,3	214	0,35	1,8	82
14	245	0,35	1,8	108

EW

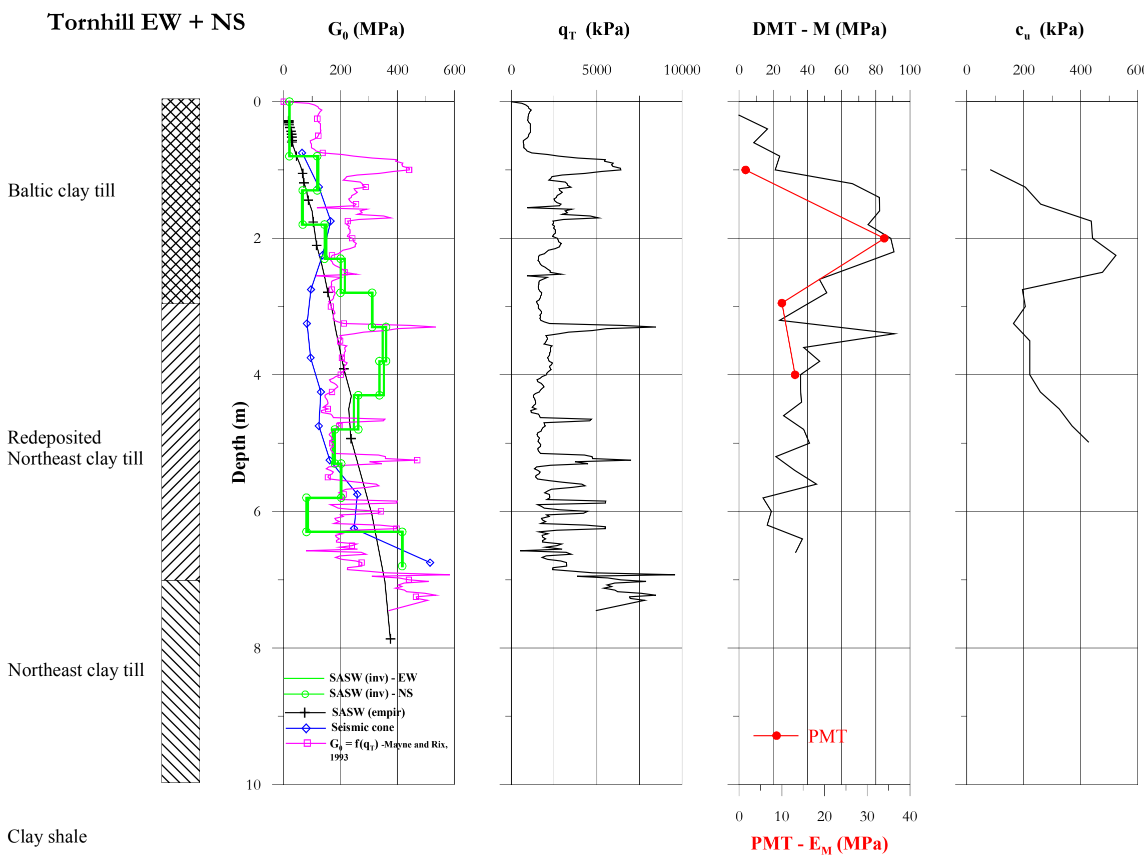
Depth (m)	vs (m/s)	Poisson's ratio	Density (t/m3)	Gmax (MPa)
0,5	87	0,5	2	15
1,5	243	1,5	2	118
2,5	145	2,5	2	42
3,5	161	3,5	2	52
4,5	236	4,5	2	112
5,5	237	5,5	2	112
6,5	151	6,5	1,85	42
7,5	232	7,5	1,85	100
8,5	147	8,5	1,85	40
9,5	235	9,5	1,85	102
10,5	198	10,5	1,8	71
11,3	244	11,3	1,8	107
14	275	14	1,8	136

Figure 22. Results and final model at Vatthammar. For better resolution see Appendix D.

For the Vatthammar site, the irregular trend of the SASW results is similar to the results at Vägverket. The SASW G_0-moduli from the two lines carried out differ in the uppermost 5 m with the EW line showing the most dramatic variations between the different levels, see *Figure 22*. The highest SASW G_0-values are found at depths of 1 m, 7 m and 9 m. A similar trend is found in the CPT results, both in terms of q_T-values and evaluated shear moduli from the seismic measurements. When compared to the SCPT moduli, the average SASW moduli follows the trend line of the SCPT moduli.

Similar to the results from the Vägverket site, it seems as if both the silty and thin layered soil profile gives a larger variation in the SASW moduli than what may be assumed to be relevant. In this case, the $G_0(q_c)$ moduli are much higher than both the SASW and the SCPT values. The empirical SASW curve follows the lowest values of the modelled SASW moduli with a smooth shape of the curve.

5.5 Tornhill



NS

Depth (m)	vs (m/s)	Poisson's ratio	Density (t/m3)	Gmax (MPa)
0,8	99	0,35	2,1	20
1,3	237	0,35	2,1	118
1,8	179	0,35	2,1	68
2,3	262	0,35	2,1	144
2,8	309	0,35	2,1	200
3,3	385	0,35	2,1	311
3,8	414	0,35	2,1	360
4,3	401	0,35	2,1	337
4,8	338	0,35	2,3	263
5,3	280	0,35	2,3	181
5,8	297	0,35	2,3	203
6,3	187	0,35	2,3	81
6,8	426	0,35	2,3	417
8	627	0,35	2,3	903

EW

Depth (m)	vs (m/s)	Poisson's ratio	Density (t/m3)	Gmax (MPa)
0,8	99	0,35	2,10	20
1,3	241	0,35	2,10	122
1,8	175	0,35	2,10	65
2,3	267	0,35	2,10	150
2,8	320	0,35	2,10	216
3,3	385	0,35	2,10	311
3,8	407	0,35	2,10	348
4,3	392	0,35	2,30	353
4,8	327	0,35	2,30	246
5,3	274	0,35	2,30	173
5,8	295	0,35	2,30	200
6,3	193	0,35	2,30	85
6,8	426	0,35	2,30	417
8	623	0,35	2,30	894

Figure 23. Results and final model at Tornhill. For better resolution see Appendix E.

The correlation between the results from the two SASW lines is good, showing a general increase of the stiffness through the Baltic clay till, down to a depth of 4 m, see *Figure 23*. The highest moduli are evaluated in the upper part of the redeposited Northeast clay till. This high moduli is not seen in the SCPT results on the same level, but indications of stiffer material can be seen in the q_T-values and DMT- results. On the other hand, the corresponding indications of a maximum in the stiffness at about 2 m depth is not found in the SASW results. A zone with lower stiffness is interpreted between 4 m and 6 m. A rapid increase of the moduli then occurs indicating the transition to the North-east clay till. The SCPT data show a low moduli zone but between 3 and 6 m depth. The empirical interpretation of the SASW data does not show any peaks or dips but a more linear increase with depth. Its smooth shape follows the average trend of the other data.

6 Discussion of the results

1) The moduli determined by SASW measurements and the inversion method and SCPT results are in broad agreement for all the sites, even if the coherence varies strongly between different types of soil profiles. There are major discrepancies between the results at certain levels in more complex profiles, for example in the silty and layered soil at the Vägverket site and in the highly variable clay till at Tornhill. On the other hand, the agreement of the two methods is better, and may even be considered as fairly good, in the clay profiles in Norrköping and Mellösa.

2) The empirical interpretation of the SASW data, where the depth related to each interpreted modulus is assumed to be half of the wavelength, seems to estimate the trend of the shear modulus with depth rather well. For the three sites Norrköping, Mellösa and Vägverket, where the stiffness increases relatively gradually and continuously with depth, the empirical moduli correlates well to the SCPT curve. However, it is not possible to obtain a similar information about its detailed variation as when using some kind of modelling tool.

3) In the silty and layered materials at the Vägverket, Vatthammar and Tornhill sites, the more advanced SASW interpretation technique appears to "over-react" when the stiffness changes rapidly over a short distance. This may be related to the silty material, the thinly layered type of soil profile, the measuring technique in the field and/or probably to a great extent be caused by the modelling software SURF, which has shown problems in handling low velocity layers.

4) When the soil volume is more homogeneous, as the clay sites appear to be, the repeatability of the SASW data is found to be good and the different test lines yield similar results. Using a maximum distance of 16 m between the geophones and a drop weight of 65 kg, a penetration depth for the Raleigh waves of 10-15 m was reached in all sites.

5) In the modelling phase of the interpretation, two different computer programs were tested – forward modelling with the software WinSASW and full inversion with the software SURF. The full inversion is generally considered as a more robust method, provided the inversion algorithm is reliable. In the beginning of the project both programs were used for interpretation of data from a few selected sites, and the same conclusion was made. Therefore only the full inversion software SURF was used for the final interpretation, apart from the Mellösa embankment site. The experience from the SURF inversion program is that in order to obtain a reliable stiffness profile from the modelling, a good a priori knowledge about the stratigraphy of the soil at start of the modelling is necessary. The reason for this is that different start models can result in different final result. The four parameters involved in the modelling procedure are; 1) the thickness of each layer, 2) the density of each layer, 3) Poisson's ratio and 4) the shear wave velocity in each layer. In this project the available reference data must be regarded as exceptionally good. Access to similar data is hardly ever the case in normal projects.

7 Conclusions

1) In general, the SASW method seems to give shear moduli values of approximately the right size and has shown to be a rapid and useful method to be applied in early stages in geotechnical investigations. However, in silty and/or layered deposits more deviating results may be obtained. For an approximate determination of the trend for the variation of the shear modulus with depth, the empirical interpretation technique is a fast and useful approach. To obtain a better resolution some kind of modelling, preferably the inversion method, has to be used.

2) To obtain a high degree of confidence in the moduli results when using inversion technique for interpretation, as much information as possible of the layering should be available. If the stratigraphy is well known as input for the model all effort will be used for minimising the uncertainties of the shear moduli determination. Of the four parameters involved in the modelling procedure - the thickness of each layer, the density of each layer, Poisson's ratio and the shear wave velocity of each layer – the density and Poisson's ratio only affect the final moduli determination with approximately 10 % (Xia et al, 1999). Therefore, if layering information is available the number of theoretically possible stiffness profiles is limited to a large extent. This will improve the confidence in the moduli results. Preferably, some sounding test and soil sampling and/or possibly some other geophysical method offering stratigraphical information should be performed for calibration of the SASW modelling tool. The most rational way of conducting the investigations would probably be to perform SCPT tests for stratigraphical information and SASW measurements for coverage of the investigated area and interconnection of the results from the SCPT tests. In this way whole sections can be reliably mapped and not only the profiles in single investigation points.

3) More experience and knowledge has to be gained regarding the numerical modelling tools. Judging from the results from the Vatthammar, Vägverket and Tornhill sites also better and more sensitive modelling techniques have to be developed in order to handle different kinds of layering and soil types.

4) There is no significant common trend in the difference between SCPT and SASW shear moduli allowing a G(SASW) / G(SCPT) relation to be determined. However, in homogeneous clay profiles the results appear to be about the same.

8 References

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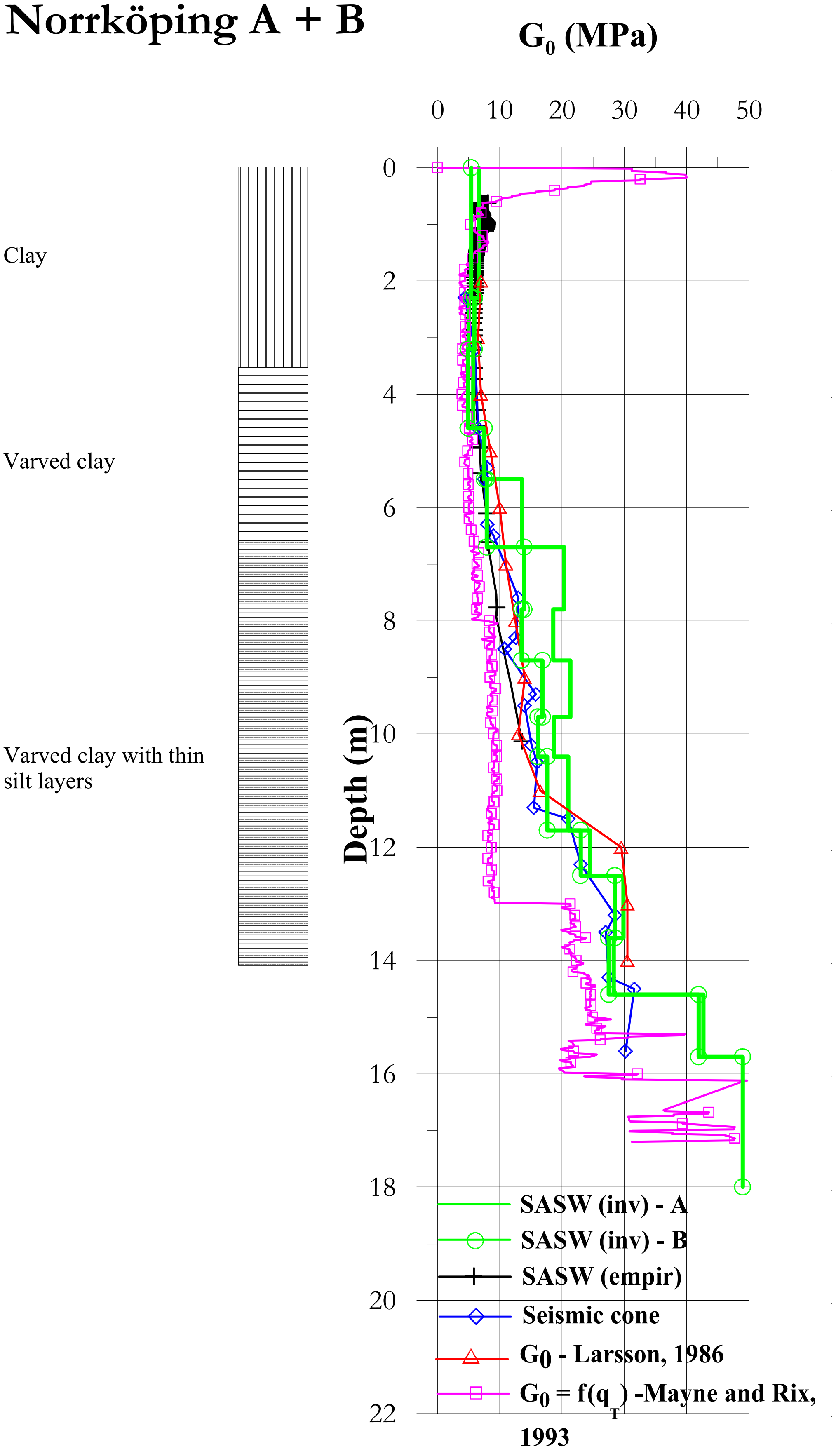
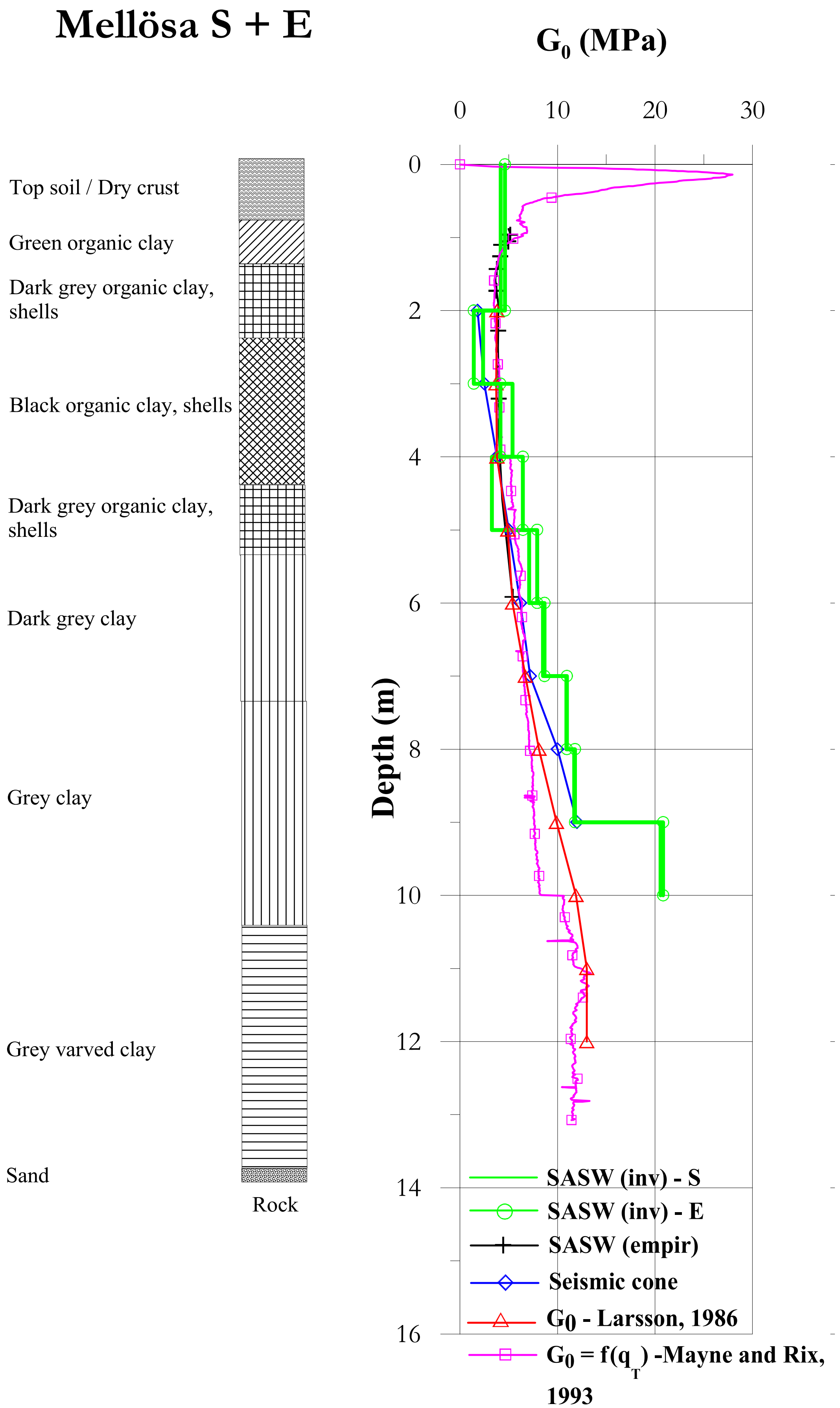
Appendix A
Results at Norrköping

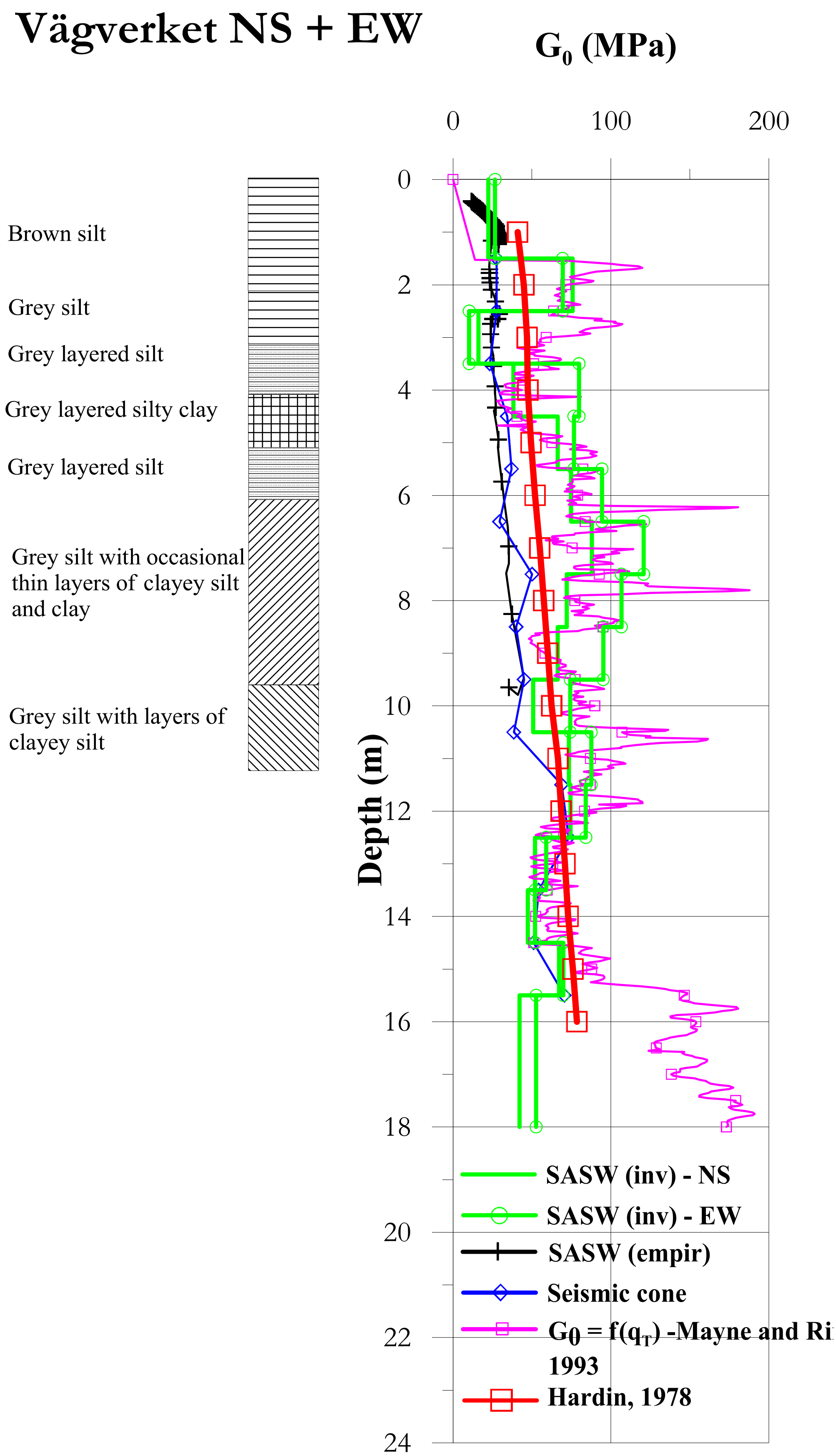
Norrköping A + B



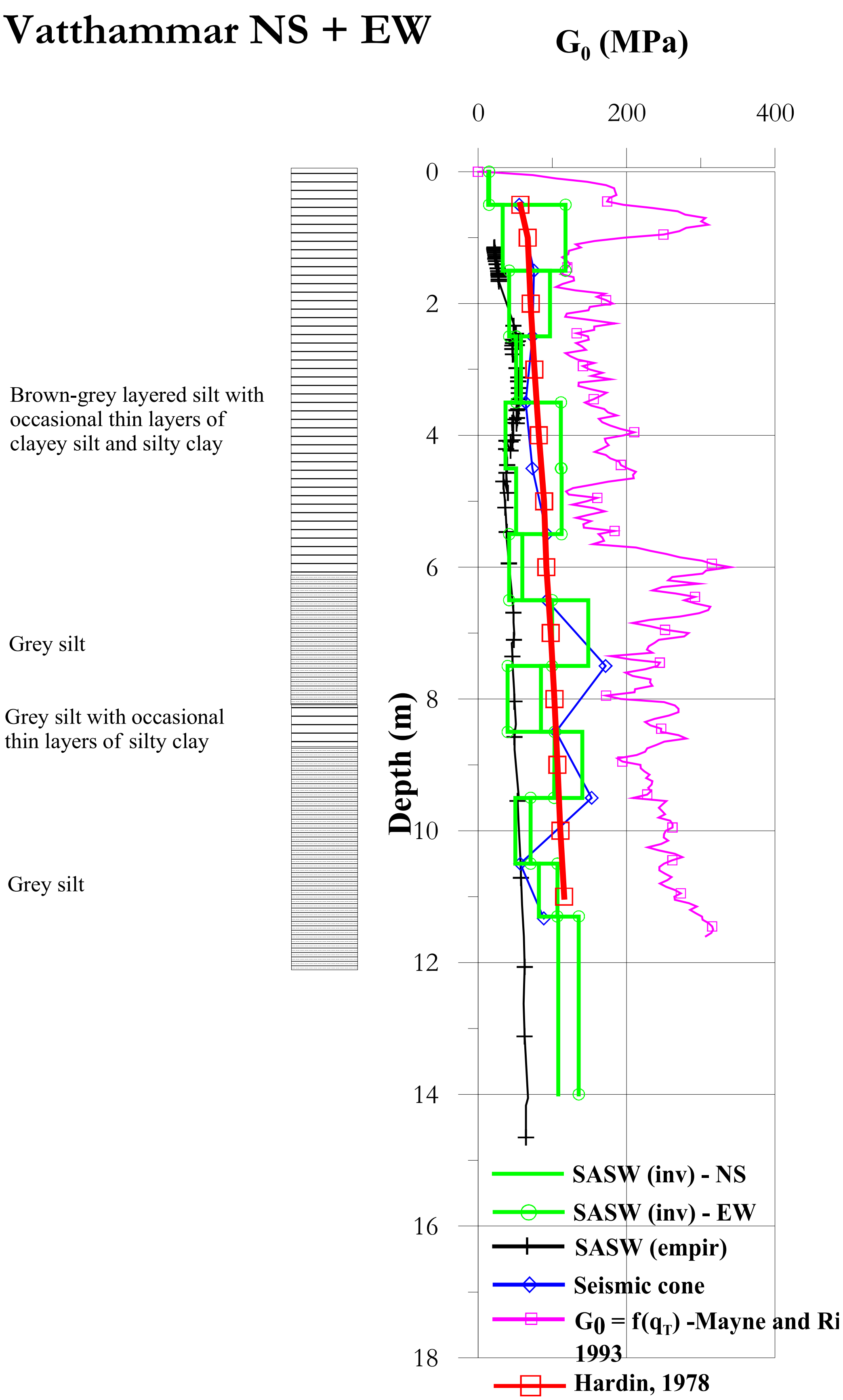
Appendix B
Results at Mellösa



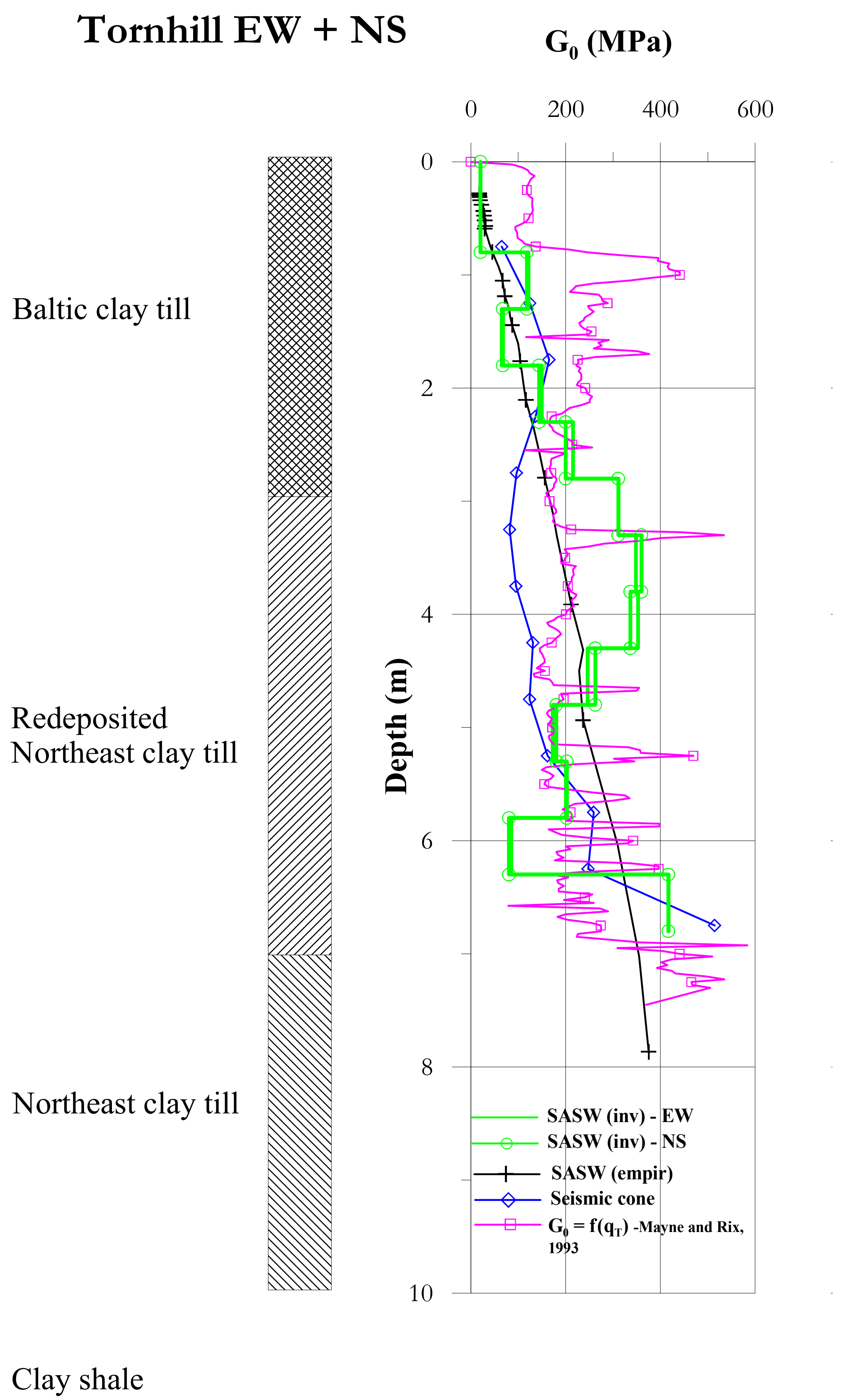
Appendix C
Results at Vägverket



Appendix D
Results at Vatthammar



Appendix E
Results at Tornhill



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