Massarsch, K. R. 2005. Deformation Properties of Stabilized Soil Columns, Proceedings, Intern. Conf. On Deep Mixing, Stockholm, 23 - 25 May, 2005. 16 p.

Deformation Properties of Stabilized Soil Columns

Massarsch K. R.
Vibisol International AB, SE-168 41 BROMMA, Sweden
rainer.massarsch@vibisol.com

ABSTRACT: The quantitative assessment of the deformation behaviour of fine-grained soils, improved by deep mixing, is important for the analysis of many geotechnical and earthquake engineering problems. Only limited information is available on the dynamic and static deformation properties, in spite of their importance for geotechnical design. The paper presents the results of extensive seismic and static investigations on soil improved by dry mixing. Seismic methods are described, which can be used in the field and in the laboratory for the determination of the shear wave velocity at small strains. The effects of strain rate and shear strain level on the soil modulus are discussed. Based on the results of extensive field and laboratory tests, recommendations are given regarding the assessment of the deformation modulus at static and dynamic loading.

1 INTRODUCTION

Deep mixing is used extensively to improve the geotechnical properties of soft soils. The most common objectives are to reduce total and differential settlements and to increase the stability of embankments and slopes. Deep soil mixing can also enhance the dynamic and cyclic properties of the ground in seismic regions. In some parts of the world, deep mixing is used extensively for the strengthening of foundations for marine and off-shore installations. Another area of increasing importance is to modify the dynamic response of dynamically loaded foundations (e.g. railway embankments or machine foundations). Surprisingly little information can be found in the geotechnical literature regarding the dynamic and cyclic deformation properties of soil, treated by deep mixing.

The realistic assessment of their properties is important for many geotechnical design problems. Figure 1 illustrates the problem of load transfer from an embankment to a soil layer reinforced by stabilized soil columns. The stress distribution between the stiffer elements and the untreated soil depends primarily on the geometric arrangement of the individual columns and on the relative stiffness of the unstabilized soil and the columns. Therefore, it is important to know as accurately as possible the deformation properties of the virgin soil, of the individual columns and of the composite structure.

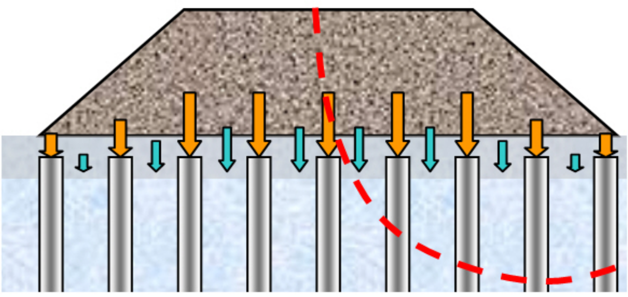


Figure 1. Load transfer from embankment to soil stabilized by columns.

The most common investigation method for deep mixing projects is laboratory testing of the soil by adding different quantities of a stabilizing agent. The increase in strength is measured at different time intervals after mixing, usually by the unconfined compression test. The undrained shear strength is often correlated with the deformation (shear and/or compression) modulus, using empirical methods. However, such correlations must be used cautiously as soil type, mixing method and curing conditions affect the results, which may not reflect the actual conditions *in situ*.

Tests on samples from actually installed columns provide more realistic information than artificially mixed samples. At the design stage of a project, it is usually difficult to establish the geotechnical properties of the improved ground by field trials. The most reliable method is by full-scale tests in the field. However, such field trials can usually be performed only in the case of large or complex projects. It is also difficult to determine the stiffness of stabilized columns with conventional geotechnical methods. Instead, push-down/pull-up probing is commonly used, from which it is difficult to assess deformation properties reliably.

Another important aspect is that the strength and stiffness of stabilized soils increases with time. Field conditions are different to those in the laboratory and it is therefore difficult to assess this effect by laboratory tests. Thus, there is a need for simple, yet reliable field and laboratory methods which can determine the static properties (deformation modulus) and the seismic properties (shear wave velocity) of stabilized columns.

In the present paper, the results of comprehensive seismic field and laboratory tests are described. The strength and deformation properties of actually installed columns were tested by conventional static tests, as well as by seismic field and laboratory tests. It will be shown that seismic methods are useful for the determination of the deformation modulus of the unstabilized soil, as well as of the stabilized columns at different time intervals after installation.

2 SEISMIC TESTING

Geophysical methods are used increasingly in geotechnical engineering, as they have several advantages compared to conventional geotechnical investigations. The most important one is that geophysical tests are non-intrusive and can thus be performed repeatedly without affecting the investigated material. They are relatively cheap and can be calibrated with geotechnical field and laboratory tests. In the present investigation, seismic testing was used in the field and in the laboratory.

2.1 Seismic Methods

Seismic methods were initially developed for earthquake applications but are used increasingly for the solution of geotechnical problems, (Stokoe & Santamarina, 2000). The seismic test is particularly suited for the investigation of soils improved by deep mixing, as the test can be performed before, and repeated at any time during and after treatment. The seismic down-hole test, which was used primarily in the present investigation, will be described in detail below.

The Seismic Analysis of Surface Wave (SASW) is a relatively recent development of seismic testing, which has found application in various areas of geotechnical engineering (Stokoe et al., 2004). It has large potential in monitoring ground improvement as the average deformation properties of a larger soil volume (i.e. the properties of soil stabilized by deep mixing or vibratory compaction) can be determined. SASW involves the active excitation of Rayleigh waves in one point and measuring the resulting vertical surface motions at various distances. Measurements can be performed at multiple receiver spacings in a linear array.

In Scandinavia, the dynamic plate load test has been used by several investigators, (Andreasson, 1979, Bodare, 1983, Massarsch, 2004).

In the laboratory, the most common seismic method is the Resonant Column Test (Woods and Henke, 1981). Undisturbed or reconstituted soil samples can be consolidated to the desired confining stress. Thereafter, shear wave velocity can be measured over a wide strain range, typically from 0.0001 – 0.1 % shear strain.

An interesting development is the bender element test, which can be combined with conventional laboratory methods, such as the triaxial and oedometer test (Dyvik & Madshus, 1985).

2.2 Seismic Down-hole Test

The seismic down-hole test, and in particular the seismic cone penetration tests (SCPT) are becoming routine investigation methods in many parts of the world (Robertson et al, 1986). The seismic down-hole test determines the travel time of polarized waves between the source on the ground surface and one or several sensor installed at different depths. As the distance between the source and the sensors is known, the compression and/or shear wave velocity can be calculated. The interpretation of the down-hole test is a straightforward procedure (Campanella et al. 1989).

The main components of the down-hole test, used for investigating lime-cement (LC) columns as reported in the present paper, are shown in Figure 2. Horizontally sensitive vibration sensors are installed either by drilling (in stiff soils) or using push-down methods. In the present investigation, purpose-built displacement-type sensors were used (Axelsson, 1996). These are relatively inexpensive and can be left in the ground after the investigation.

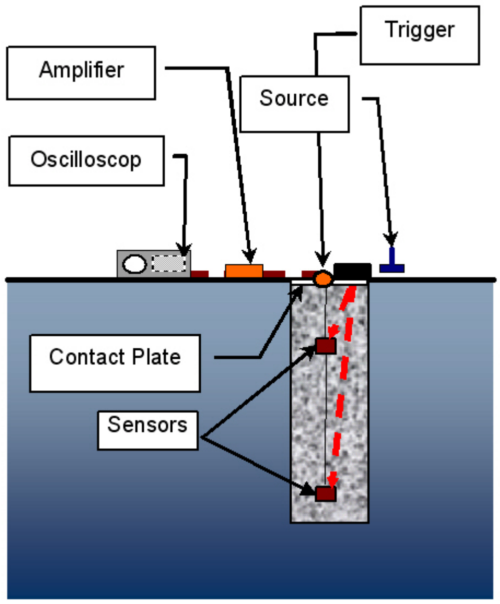


Figure 2. Principle of seismic down-hole test in LC column.

An impulse is generated by an energy source (usually a hammer), striking a plate firmly in contact with the top of the column or anchored to the ground surface. Usually, horizontally polarized shear waves are generated. The start of the propagation of the wave is recorded by a trigger. The arrival time of the wave at different locations below the ground surface is measured by vibration sensors. The signal is amplified and recorded by an oscilloscope. The test is usually repeated, reversing the direction of polarization.

2.3 Waves

Seismic methods measure the wave propagation velocity in a material from which the low-strain modulus of the material can be calculated. Two types of body waves can be used for seismic tests, compression waves (P-waves) and shear waves (S-waves). The compression wave travels faster and arrives thus first at the observation point. The shear wave is slower but has the important advantage that its propagation velocity is not affected by ground water. Also, due to the lower propagation speed, the shear wave velocity can be measured with greater accuracy, as the time interval is larger than in the case of P-waves. In the case of down-hole testing, usually only the shear wave velocity is measured.

2.4 Interpretation of Test Results

The objective of the seismic down-hole test is to determine the travel time of the polarized shear wave between different sensor levels. If the distance is known, the shear wave velocity can be calculated. Different methods exist to determine the travel time. The chosen method can affect the accuracy and reliability of the test results and some experience is required. The different evaluation methods are discussed briefly below.

First Arrival Method

The first arrival method is subjective as the travel time between the respective transducer locations is determined in a visual manner. Figure 3 shows vibration records of two geophones located at a distance of 2 m. The travel time can be identified at different points in the vibration record. Point 1 corresponds to the “first arrival” of the shear wave (first off-set from the zero line). Point 2 is at the first peak (trough) of the propagating wave and represents the propagation velocity at which the shear wave energy travels. Point 3 represents the second crossing of the signal of the zero line and is sometimes easier to identify than the first arrival. If the quality of the signal is good, all three methods give approximately the same wave propagation velocity.

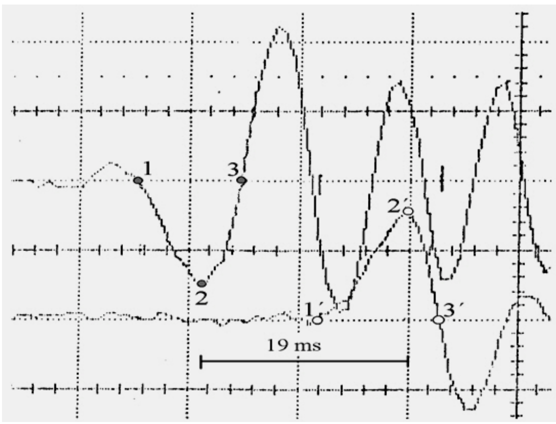


Figure 3. Determination of the first arrival time at different time intervals.

In Figure 3, for all three cases, the time interval is approximately 19 ms, which at a sensor spacing of 2 m corresponds to a shear wave velocity of 105 m/s. The predominant frequency of the wave can be estimated from the period of the signal. The wave length, L can be obtained form the following relationship

$$L = C / f \tag{1}$$

where C is the wave velocity and f is the frequency. This information is useful as it indicates the volume of tested soil.

Reverse Impact Method

In order to simplify the evaluation of the seismic test, it is recommended to perform two blows in opposite directions (“reverse impact method”), cf. Figure 4. The curves show vibration records at two depths. The reverse impact method facilitates the identification of travel times and its use is recommended.

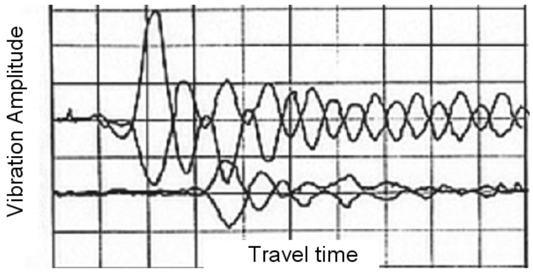


Figure 4. Reverse impact test of seismic down-hole test for determination of first arrival time.

Cross-correlation

Cross-correlation determines the time interval between two signals, based on the maximum signal amplitude. Signal evaluation by this method is less subjective, but requires judgement as the result are influenced by the part of the time history is selected for analysis. The signal amplitude of the two transducers is adjusted to the same value. Cross-correlation shifts the second signal relative to the first one, and shifting is performed in steps equal to the time interval between the digitized points. At each time shift, the sum of the product of the two signal amplitudes is calculated. This sum is plotted against the time shift, and the time shift at the maximum value of this graph is the time difference between the arrival times of the transducers, Figure 5. The peak value of the cross-correlation represents the propagation velocity of the wave energy.

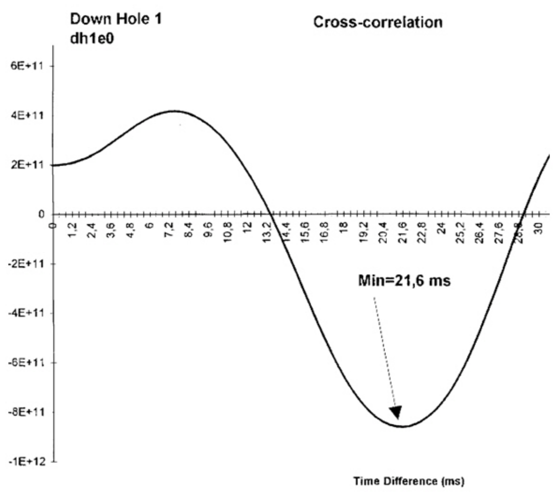


Figure 5. Determination of travel time between sensors using the cross-correlation method.

2.5 Dynamic Soil Properties

The primary result of a seismic investigation is the wave propagation velocity of either P-waves or S-waves. From the P- and S-wave velocities the shear modulus, Gmax and the oedometer (constrained) modulus, M_{max}, at small strains can be calculated from the following relationships

$$G_{max} = \rho\, C_S^2 \tag{2}$$

$$M_{max} = \rho\, C_P^2 \tag{3}$$

where ρ is the bulk density of the soil and Cp and Cs are the P-wave velocity and S-wave velocity, respectively. Shear waves propagate at very low strains, and the shear strain level γ can be estimated from the following relationship,

$$\gamma = \frac{x}{C_S} \tag{4}$$

where x is the vibration velocity amplitude. If for instance the vibration amplitude is 0.1 mm/s and the shear wave velocity is 100 m/s then the shear strain level is 10^{-4} %. At such a low strain level, the soil is in the elastic range.

It is often assumed that the rate of deformation during a dynamic test is high. This is not correct, as the strain amplitude is very low. Consequently, the strain rate of a seismic test is comparable to that of a static test, Massarsch (2004). Thus, the reason for the difference between the "seismic modulus" and the "static modulus" is that the seismic modulus is determined at a much lower strain level than the static modulus.

3 DEFORMATION PROPERTIES OF FINE-GRAINED SOILS

3.1 Deformation Modulus

The shear modulus at small strain (approx. 0.001 %) can be determined from seismic tests in the field or from resonant column tests in the laboratory. From the shear wave velocity, the shear modulus Gmax at small strains can be calculated according to Eq. 2. Figure 6 shows a typical shear stress-shear strain relationship.

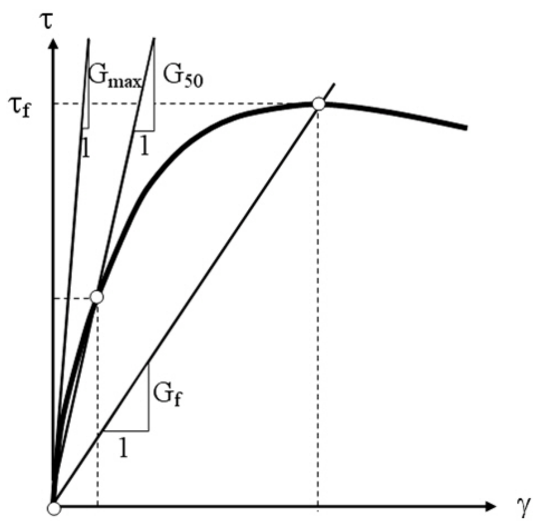


Figure 6. Shear stress – shear strain relationship for fine-grained soil at undrained loading.

Three commonly used definitions of the shear modulus G are indicated. At very low stress levels (i.e. very low strains), the shear modulus is called the maximum shear modulus, G_{max}. With increasing stress level, the shear modulus decreases. At 50 % of the failure stress the term G_{50} is frequently used, which corresponds to typical operating conditions (service state) of a geotechnical structure. At failure, the shear modulus is defined as G_f. It is common practice to define the stress-strain relationship of soils by the secant modulus, G_s. At unloading and re-loading, it is usually assumed that the modulus corresponds to the modulus at initial loading, G_{max}. The measuring accuracy of conventional static laboratory tests has improved and stress-strain measurements can today be performed at very low strain levels, during triaxial, simple or direct shear tests. It is possible to estimate the shear modulus quite accurately at strain levels above about 0.5 %.

3.2 Correlation between Gmax and τ_{fu}

For normally consolidated, fine-grained soils, a close correlation exists between the ratio $\tau f\ /\sigma_v'$ and the plasticity index, PI (Bjerrum, 1973)

$$\frac{\tau_{fu}}{\sigma_v'} = 0.0029PI + 0.13 \tag{5}$$

where σ_v' is the vertical effective stress. Hardin (1978) has proposed the following semi-empirical relationship for estimating the shear modulus at small strains, Gmax

$$G_{max} = \frac{625}{0.3 + 0.7e^2} OCR^k \left(\sigma_0' p_a\right)^{0.5} \tag{6}$$

where e = void ratio, OCR = overconsolidation ratio, k = empirical constant, which depends on PI, σ_0 is the mean effective stress and pa is a reference stress (98.1 kPa). The shear modulus at small strains is thus a function of the square root of the mean effective stress. Therefore, the assumption of a linear relationship of G/τ_f appears not to be justified. The relationship between the shear modulus at small strain, Gmax and the undrained shear strength. τ_{fu} (shear modulus ratio) for normally consolidated, saturated, fine-grained soils can be estimated from the following relationship (Massarsch, 2004),

$$\frac{G_{max}}{\sqrt{\tau_{fu} p_a}} = \frac{625}{0.3 + 0.7\left(w_n \dfrac{\rho_s}{\rho_w}\right)^2} \sqrt{\frac{1 + 2K_0}{3\left(0.0029PI + 0.13\right)}} \tag{7}$$

where w_n = natural water content, ρ_s = density of solid particles and ρ_w = density of water. K_0 is the coefficient of lateral earth pressure (effective stress) at rest, which in normally consolidated clay deposits can be estimated with sufficient accuracy from the following relationship, (Massarsch, 1979)

$$K_0 = 0.0042PI + 0.44 \tag{8}$$

The normalized shear modulus as calculated from Eq. (7) is shown in Figure 7 as a function of the water content, for different values of the plasticity index, *PI* and normally consolidated conditions (*OCR* = 1).

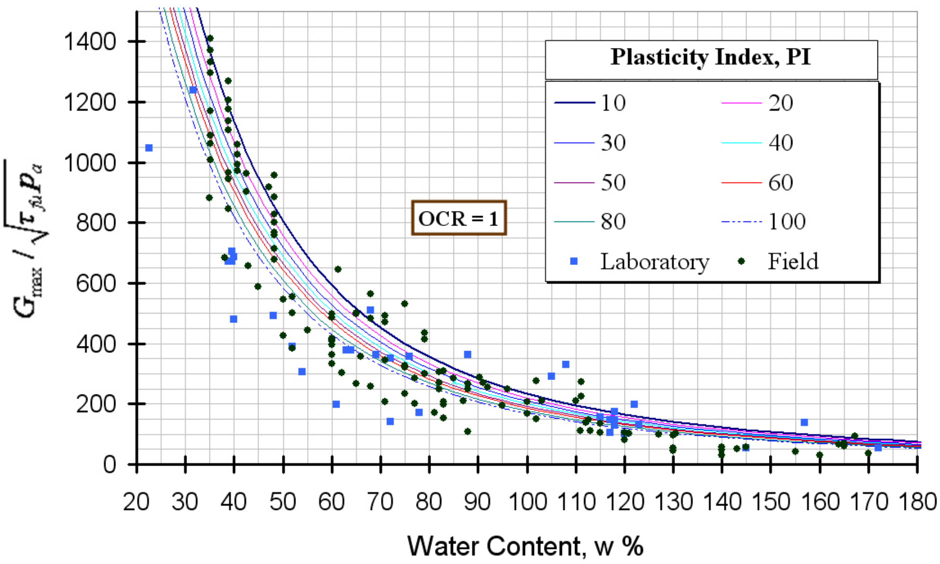


Figure 7. Relationship between the normalized shear modulus at small strains, G_{max} and the water content, cf. Eq.7; field and laboratory data from Döringer (1997).

It is assumed that the soil is fully saturated. The normalized shear modulus decreases markedly when the water content of the soil increases. In Figure 7, results of investigations published in the literature are presented. The data included field and laboratory tests and in a variety of fine-grained soils. They follow quite closely the semi-empirical relationship from to Eq. 7. Values determined in the field are generally about 10 to 20% higher than laboratory results. For many design problems, Eq. 7 can be used with sufficient accuracy.

It is apparent that water content (and the thus void ratio) has a strong influence on the small-strain modulus. The shear modulus ratio is high in silty clays and silts and can range from 1500 – 2000. In the case of low-plastic clays (w_n around 20 %), the ratio is in excess of 1000, but decreases to 250 in plastic clays with w_n approaching 100 %. The modulus ratio can be even lower in organic soils, but the available database is limited.

The following example illustrates the use of Figure 7. Assuming a normally consolidated soft clay, with τ_{fu} = 15 kPa, w_n = 50 %, and PI = 40, the shear modulus ratio, $G_{max}/(\tau_{fu}p_a)^{0.5}$ = 750. The shear modulus at small strain is thus, G_{max} = 28.7 MPa. In this case, the rigidity index (ratio between the shear modulus and the undrained shear strength), G_{max}/τ_{fu} = 1920. The elastic modulus E and the constrained modulus M, are related to the shear modulus G according to the following relationships

$$E = 2\,(1+\nu)\,G \tag{9}$$

$$M = \frac{2\,(1-\nu)}{(1-2\nu)}\,G \tag{10}$$

where ν is Poisson's ratio. It is commonly assumed that for undrained conditions in fine-grained soils, ν = 0.5, which is valid for undrained conditions at large strains (> 0.1 %). However, this value is not necessarily valid at small strain levels (< 0.001 %), where ν can be significantly lower (0.15 – 0.3). This aspect can have important practical consequences when interpreting the results of small-strain tests, but it is usually not appreciated.

3.3 Shear Modulus at Large Strains

The stress-strain behaviour of fine-grained soils has been investigated extensively in the areas of soil dynamics and earthquake engineering. The most widely used correlation was proposed by Vucetic and Dobry (1991). Massarsch (2004) analyzed stress-strain data published in the literature (mainly RC tests) and performed a regression analysis. A modulus reduction factor, Rm = Gs/Gmax was used to define the decrease of the shear modulus Gs at three shear strain levels, 0.1, 0.25 and 0. 5 %, cf. Figure 8. The modulus reduction factor Rm decreases rapidly in the case of silty soils, and less in soils with higher PI.

For example, in a soil with PI = 20 % at γ = 0.1 %, the shear modulus is 0.45 G_{max}. At a shear strain level of γ = 0.5 %, the modulus value decreases to 0.15 G_{max}. For normally consolidated clay with a plasticity index of 40%, the modulus reduction factor at 0.5% shear strain, R_m = 0.28. Thus the maximum shear modulus, G_{max} (28.7 MPa) decreases at a stress level of approximately 50 % of the failure load to G_{50} = 8.1 MPa. Assuming Poisson's ratio ν = 0.5, according to Eq. 9, the elastic modulus E_{50} = 24.3 MPa. The ratio between the elastic modulus E_{50} (24.3 MPa) and the undrained shear strength (20 kPa) is thus approximately E_{50}/τ_{fu} = 1200. The shear strain γ at G_{50} can be estimated from the ratio of the shear modulus and the undrained shear strength: $\gamma = \tau_{fu}/G$ (20/8100 = 0.00185) and is about γ_{50} = 0.25 %.

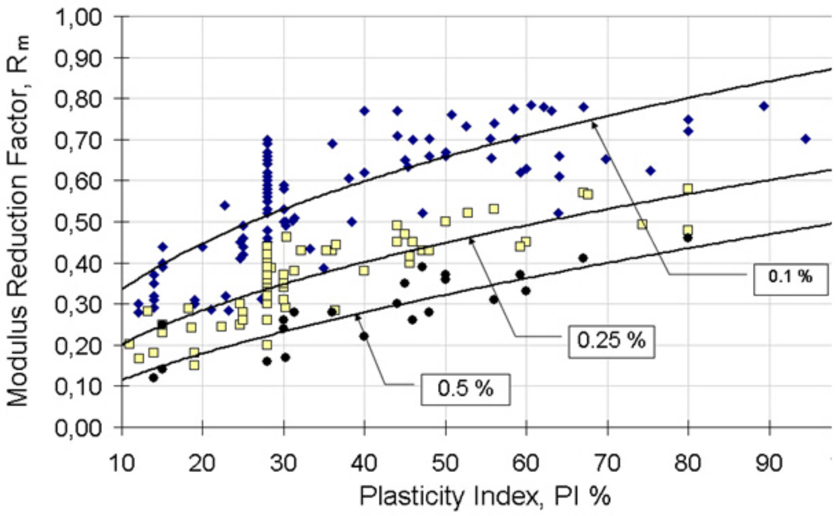


Figure 8. Modulus reduction factor, $R_m = G_s/G_{max}$ as function of the plasticity index, *PI* at three strain levels, (Massarsch, 2004).

4 DEFORMATION PROPERTIES OF LIME-CEMENT COLUMNS FROM STATIC TESTS

4.1 Static Laboratory Tests

Only limited information is available in the literature concerning the deformation properties of LC columns during undrained loading. Massarsch and Eriksson (2002) compiled the results of field and laboratory tests, Figure 9, where the elastic modulus at 50 % of the failure load is shown as a function of the compressive strength.

The scatter in the data is relatively large. The modulus values from laboratory tests are about 2 to 3 times higher than those determined by in-situ tests. The following correlation was obtained between the unconfined compressive strength (kPa) $q_{u,col}$ and the modulus of elasticity (MPa) E_{50} at 50 %.

$$E_{50} = 160\, q_{u,col} \tag{11}$$

4.2 Static Field Tests on LC Columns

Kivelö (1994) reported static field loading tests on individual LC columns. The LC columns were installed in soft, plastic clay with undrained shear strength of 18 – 20 kPa and water content 43%. The mixing ratio of cement and unslaked lime was 50% (20 – 25 kg/m), and the column diameter was 0.5 m (corresponding to 102-127 kg/m^3). Deformations at different depths in the column were measured as the load on the column head was increased. The reduction of the elastic modulus as a function of the applied load was measured. The elastic modulus E of the upper part (1 m of column length), normalized by the initially modulus value is shown in Figure 10 as a function of strain. The modulus reduction at 0.5 % axial strain was about 50 % of the maximum value. However, the maximum value, which was used to normalise the elastic modulus, was probably lower than a maximum value based on a seismic tests (Gmax). However, it is apparent that the elastic modulus decreases significantly with increasing strain level.

5 NORRALA PROJECT

5.1 Project Site

In connection with the expansion of the Ostkustbanan railway link north of Uppsala, Sweden, an up to 7 m high embankment had to be constructed on very soft, compressible soil. The main objective of the project is to increase the train speed from 130 to 160 km/h. The most common ground improvement method for such problems in the Nordic countries is the installation of LC columns, using the dry mixing method. While different methods can be used to determine the geotechnical parameters of the columns, only limited data is available on their static and dynamic deformation behaviour. Therefore, extensive field and laboratory investigations were implemented at Norrala (Axelsson, 1996, Björkman and Ryding, 1996).

The test area consisted of soft, organic clay to a depth of 10 m. Below a 1 m thick dry crust follow: 2 m of organic clay (gyttja), 2 m of sulfide clay (locally known as “svartmocka”) and 4 m of silty clay. Moraine (till) was encountered at 9 m depth. The ground water level was located less than 1 m below the ground surface. The soil conditions are summarized in Table 1, where the water content, the density, the undrained shear strength and the sensitivity are given.

The shear strength was determined by the fall cone test and is usually corrected with respect to *PI* (Bjerrum, 1973). Assuming *PI* = 60, the correction factor is approximately 0.75.

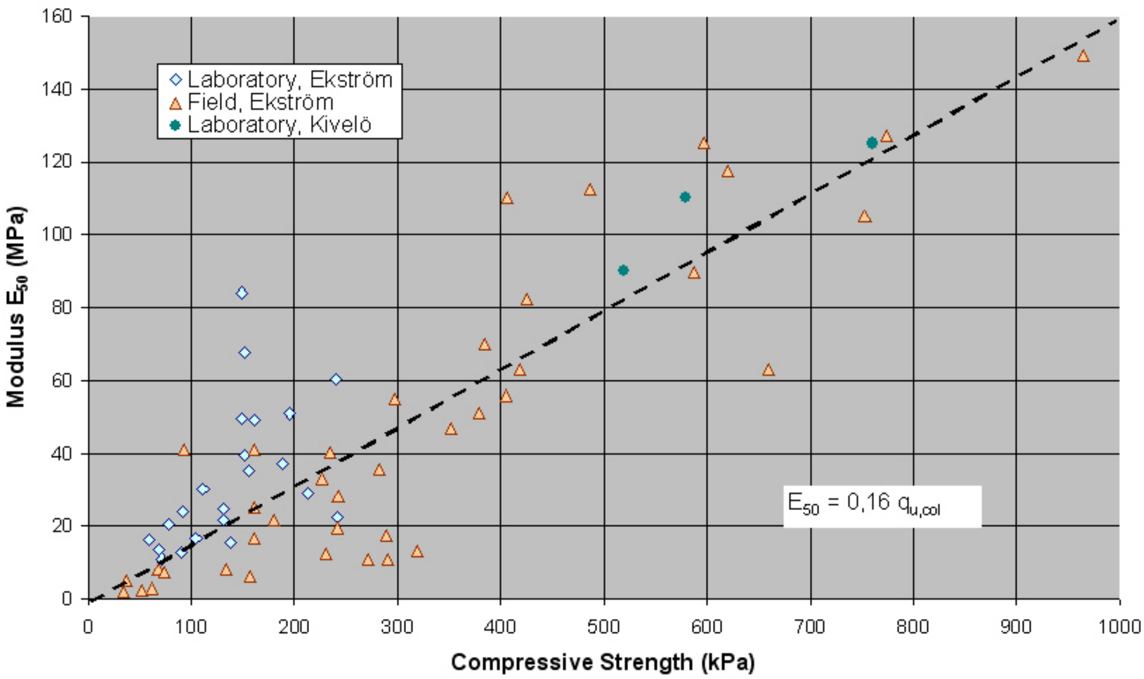


Figure 9. Relationship between the elastic modulus, E_{50} and the unconfined compressive strength from LC columns (Massarsch and Eriksson, 2002).

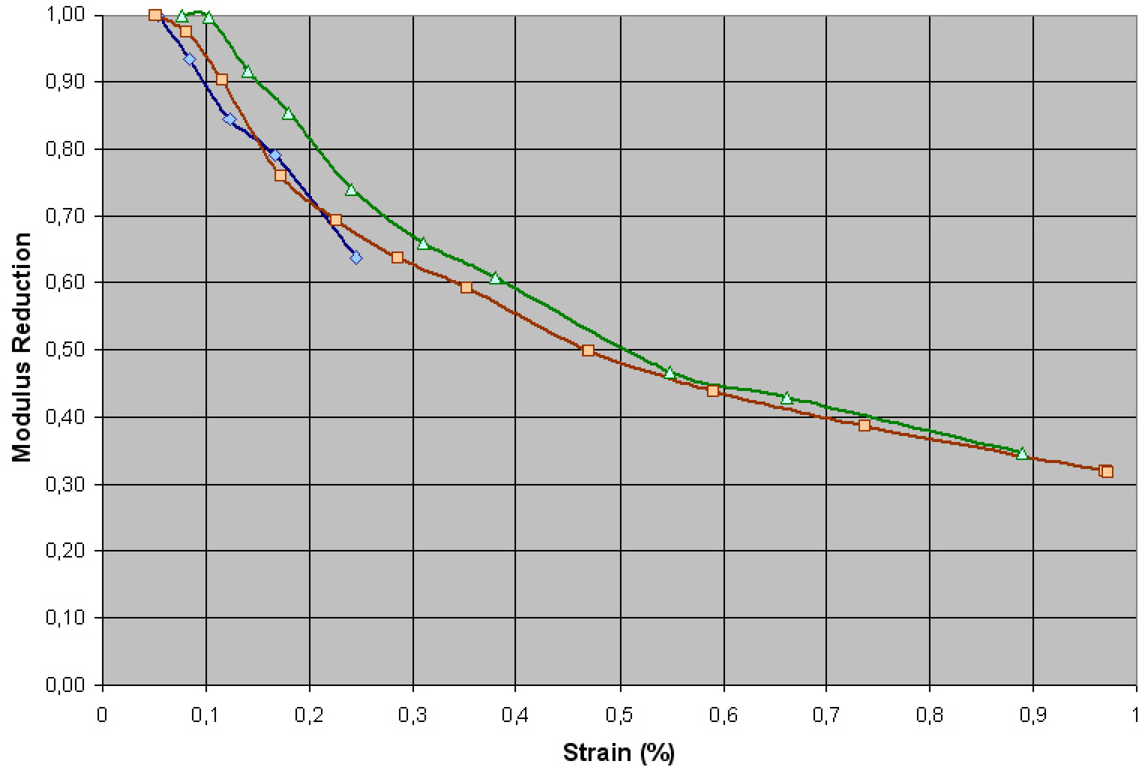


Figure 10. Reduction of compression modulus from field load tests on LC columns, after Kivelö (1994).

Table 1. Geotechnical conditions at Norrala test site.

Depth	Soil	Water content	Density	τ_{fu}	S_t
(m)		(%)	(t/m^3)	kPa	-
0	Dry crust	-	1.2	-	-
1	Gyttja	107	1.4	12	11
2	Gyttja	81	1.4	13	17
3	Gyttja	124	1.38	20	7
4	Svart-mocka	87	1.4	17	20
5	Svart-mocka	74	1.4	17	20
6	Svart-mocka	74	1.4	17	20
7	Silty clay	50	1.7	9	16
8	Silty clay	50	1.7	9	16
9	Silty clay	50	1.7	15	16
10	Moraine	-	1.9	-	-

5.2 Tests on LC Columns

On June 6, 1995 (from 12.00 – 13.00) 18 LC test columns of 10 m length and 0.60 m diameter were installed. The mixing ratio of lime/cement was 50% / 50%. Two mixing quantities were used: 28 kg/m (99 kg/m^3) and 44 kg/m (156 kg/m^3), respectively. Inspection of the samples and of several test columns showed that the columns were relatively homogeneous. However, the lime-cement content varied across the column cross section.

Static Tests in the Laboratory

Samples from LC columns with different quantities of stabilizing agent (28 kg/m and 44 kg/m) were obtained by coring, and investigated by triaxial and direct shear tests (Björkman and Ryding, 1996). The samples were taken at a distance of 0.12 m from the edge of the column. All samples were obtained from the same depth, 3 m below the ground surface. The laboratory tests took place during September and October 1995, about 4 months after installation of the test columns. Isotropically consolidated drained (CD) and undrained (CU) triaxial tests were conducted on samples with a diameter of 49 mm and a height of 100 mm. The samples were first consolidated isotropically to in-situ stresses and thereafter loaded to failure. The loading rate was 0.1 mm/min, which is comparable to that of a seismic test (0.002 %/sec). In the present paper, only the results from the CU tests are reported. Details of the testing procedure and results from all tests have been published by Björkman and Ryding, (1996).

Direct shear tests were also performed in a shear box (“Mulbert”) with a sample diameter of 150 mm and a sample height of 50 mm, at three different vertical pressures. The loading rate during the shear tests was 0.5 mm/min and failure was typically reached within 2 minutes.

Seismic Field Tests

Immediately after installation of the columns, and before the start of the curing process, vibration sensors were installed at 2.5 and 5.5 m depth, respectively. In addition, at some distance from the test columns, two sensors were installed in the clay at 2 and 5 m depth, respectively. Details of the measuring equipment, the installation method and evaluation techniques have been described by Axelsson (1996). Four hours after installation of the LC columns, the first down-hole test was performed according to the procedure described above. In total six down-hole tests were carried out over a time period of 41 days.

Seismic Laboratory Tests

A sample from a column with a mixing quantity of 28 kg/m (99 kg/m^3) was obtained from 3 m depth. An undisturbed sample was also taken at 3 m depth from the unimproved soil in the vicinity. Bender element tests were performed in a triaxial testing device 116 days after the installation of the test columns. The samples were first consolidated at the in-situ confining stress, during which the shear wave velocity (shear modulus) was determined at different time intervals.

5.3 Results from Static Tests

Triaxial Tests on Clay

Standard CU tests were carried out on undisturbed samples at a confining stresses of 60 kPa. The stress-strain curve is shown in Figure 11. Failure was reached at approximately 2.5 % axial strain at a deviatoric stress of 40 kPa, which corresponds to an undrained shear strength of 20 kPa.

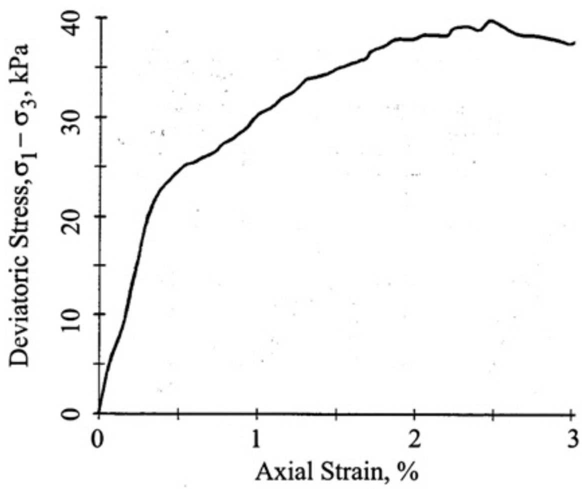
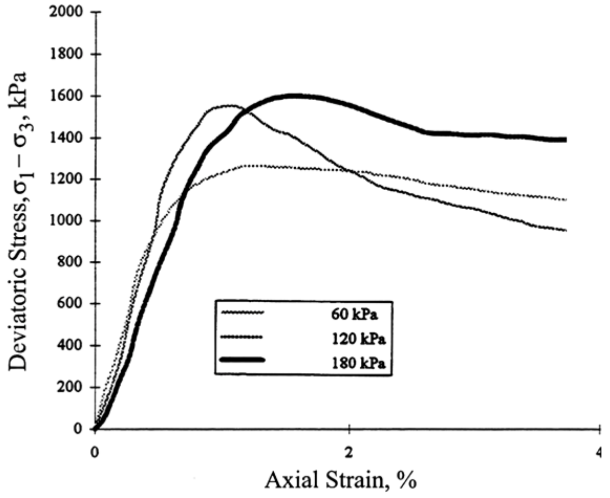


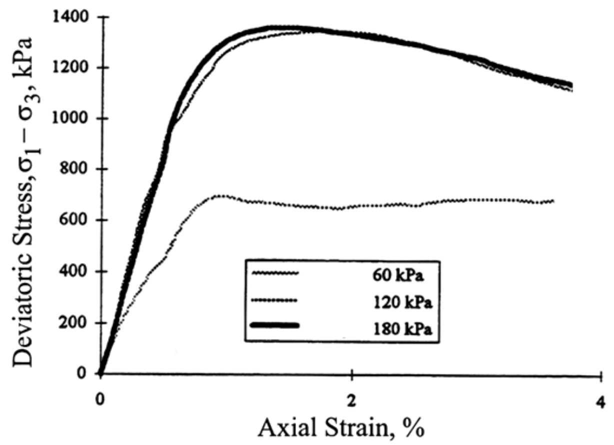
Figure 11. Triaxial undrained shear test on clay.

Triaxial Tests on LC Columns

Triaxial tests were carried out on samples with two mixing quantities (28 and 44 kg/m) at three confining stresses (60, 120 and 180 kPa). Stress-strain curves from the two sets of tests are shown in Figure 12. From the stress-strain curves it is possible to determine the elastic modulus at different strains. It should be noted that the properties of the individual samples varied considerably, in spite of the apparent homogeneity of the columns. This aspect is important when interpreting the results from samples, obtained from LC columns installed in the field. The scatter of data reflects thus the variability of the columns.



a) Mixing quantity: 28 kg/m (99 kg/m^3)



b) Mixing quantity: 44 kg/m (156 kg/m^3)

Figure 12. Stress-strain curves from CU Triaxial tests.

In Table 2, the deviatoric stress at failure, the axial strain and the elastic modulus at failure, E_f as well as the axial strain and modulus at 50 % of the failure load, E_{50} are given for three different confining stresses and two mixing quantities, respectively. It is interesting to note that no clear difference in neither strength nor soil stiffness can be detected for the two mixing quantities. This is somewhat surprising, considering the relative homogeneity of the samples.

Table 2. Determination of modulus from triaxial tests.

a) Mixing Quantity: 28 kg/m (99 kg/m^3)

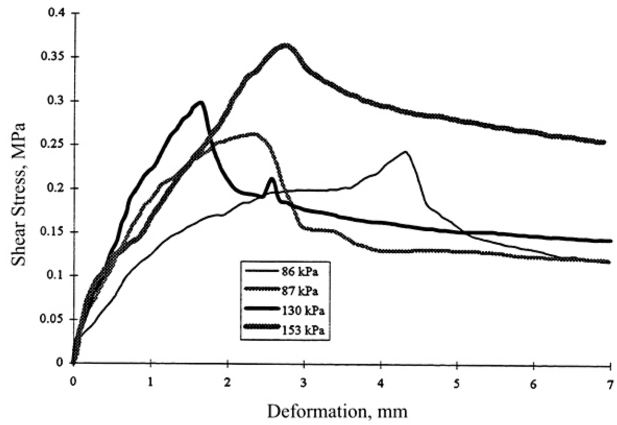
σ_3	$(\sigma_1-\sigma_3)_f$	ε_f	E_f	ε_{50}	E_{50}
k	kPa	%	MP	%	M
6	1265	1,	103	0,	2
1	1550	1,	150	0,	2
1	1600	1,	101	0,	1

b) Mixing Quantity: 44 kg/m (156 kg/m^3)

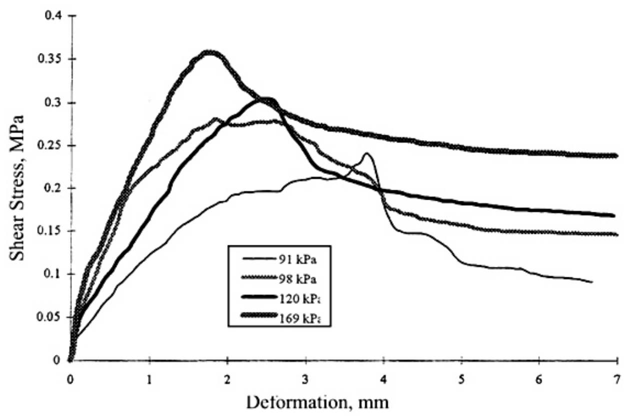
σ_3	$(\sigma_1-\sigma_3)_f$	ε_f	E_f	ε_{50}	E_{50}
k	kPa	%	M	%	M
6	695	0,	72	0,	17
1	1345	1,	73	0,	16
1	1365	1,	94	0,	10

Direct Shear Tests on LC Columns

Direct shear tests were performed on larger samples than those used for the triaxial tests. In the case of a direct shear test, the failure plane is pre-determined. The samples were inspected after each test and it was found that variations of material properties along the failure plane influenced the results. Typical results for two mixing quantities are shown in Figure 13.



a) Mixing quantity: 28 kg/m (99 kg/m^3)



b) Mixing quantity: 44 kg/m (156 kg/m^3)

Figure 13. Stress strain curves from direct shear tests.

5.4 Results from Seismic Tests

Two types of seismic tests were performed: down-hole tests in the field on the unimproved soil and in LC columns, as well as bender element tests in the laboratory on an undisturbed clay sample and on a sample from a column with a mixing quantity of 28 kg/m (99 kg/m^3).

Seismic Down-hole Tests in Clay

Seismic down-hole tests were carried out in the undisturbed clay, adjacent to the test area where the LC columns were installed. The shear wav velocity was measured at 2.5 and 5.5 m depth. The average shear wave velocity of several measurements was 40 m/s. The predominant frequency of the signal was 20 Hz, corresponding to a wave length of 2 m. The shear wave velocity gave a small strain modulus G_{max} = 2.3 MPa.

Bender Element Tests on Clay Samples

Bender element tests were performed on undisturbed samples of clay from 3 m depth. The water content prior to consolidation was 124 %. The undrained shear strength obtained from CU tests was 19.5 kPa. The samples were consolidated isotropically to a confining pressure of 8.7 and thereafter to 9.3 kPa. The shear modulus was determined during consolidation over a period of 2500 minutes, Figure 14.

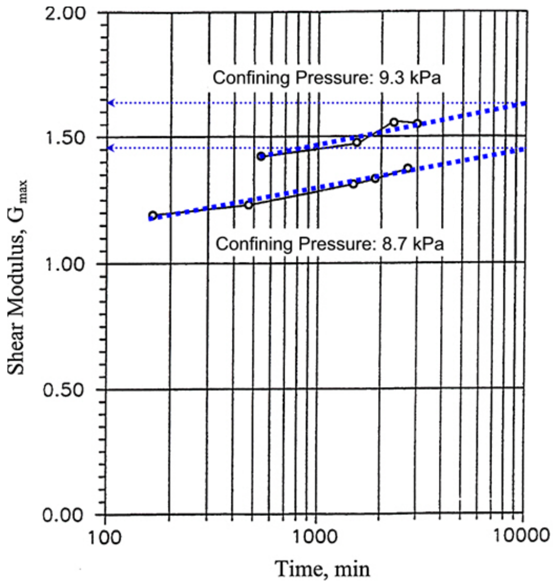


Figure 14. Shear modulus determined on clay sample from 3 m depth during consolidation at two different confining stresses.

The shear modulus increased during consolidation and had not yet reached the final value when the tests were ended. For the respective confining stress, the shear modulus at small strain, G_{max}, at the end of consolidation is 1.42 MPa and 1.79 MPa. For the analysis, an average value of the maximum shear modulus at the end of consolidation of 1.61 MPa was chosen, which corresponds to a shear wave velocity of 34 m/s. This value is significantly lower than the shear wave velocity measured in-situ (40 m/s) and confirms that consolidation was not yet completed.

Seismic Down-hole Tests in LC Columns

Seismic down-hole tests were carried out at different time intervals after installation of the LC columns. Figure 15 shows the shear wave signal in a column with mixing quantity 44 kg/m (156 kg/m^3), 41 days after installation. The measuring points are located at 2.5 and 5.5 m depth. Two tests with polarized signals in the opposite direction are superimposed.

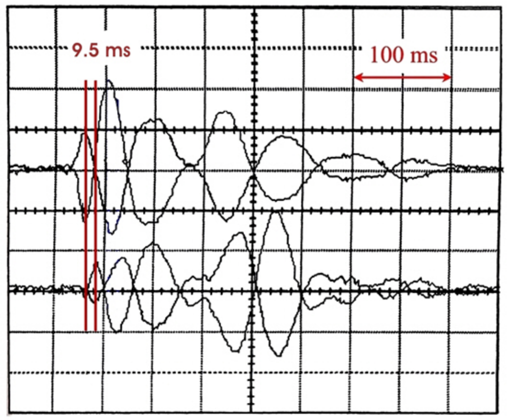


Figure 15. Signal from reverse impact test (depth interval 2.5 to 5.5 m) in column with mixing quantity 44 kg/m (156 kg/m^3), 41 days after installation.

It is relatively easy to identify the arrival of the first (or second) peak of the shear wave. However, it is not equally easy to determine the first arrival of the shear wave. The time interval between the first peak of sensor 1 and sensor 2 is 9.5 ms, resulting in a shear wave velocity of 316 m/s. A more accurate and consistent method of determining the travel time is by cross-correlation of the two shear wave signals, Figure 16.

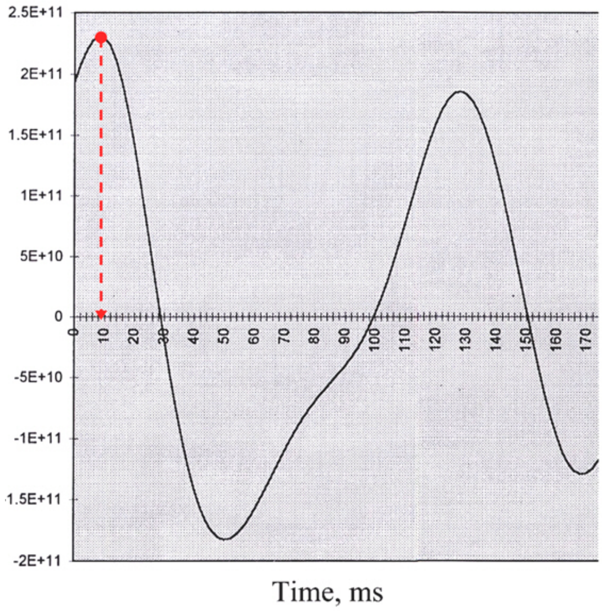


Figure 16. Determination of the shear wave velocity by cross-correlation of the signals shown in Figure 15.

There is good agreement between interpretation methods, both of which give a travel time of 9.5 ms. The predominant period of the shear wave is 72 ms, which corresponds to a frequency of 14 Hz. Assuming a shear wave velocity of 316 m/s, the wave length is 23 m, which is significantly longer than in the clay. From the shear wave velocity, and assuming a density of 2 t/m^3, the small-strain modulus 41 days after construction is G_{max} = 199 MPa.

Bender Element Tests in LC Columns

The shear modulus was also determined by bender element tests in the laboratory, 116 days after construction of the columns. A sample was taken at 3 m depth from a column with a mixing quantity of 28 kg/m. The water content of the sample was 97 % prior to consolidation. The sample was consolidated isotropically in steps at two confining stresses, lasting a time period of 1500 and 5500 minutes, respectively. Thereafter, the shear strength was determined by conventional triaxial test, which yielded $(\sigma_1 - \sigma_2)/2 = 640$ kPa. The results of the bender element test are shown in Figure 17.

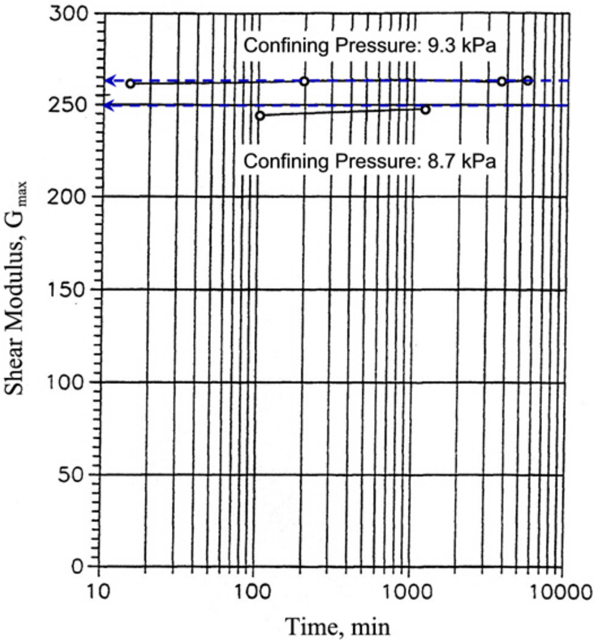


Figure 17. Results from bender element tests on consolidating sample from LC column.

The bender element tests suggest that practically no change of the shear modulus at small strain occurred during consolidation (over a time period of almost 5500 minutes). The maximum shear modulus at the two confining stresses was 250 MPa and 269 MPa, respectively. For the following evaluation of test data, an average value of 255 MPa was chosen as the maximum shear modulus. This value is about 30 % higher than the shear modulus, determined in the field by down-hole tests 41 days after installation (199 MPa). The time aspect will be discussed below in more detail.

6 DEFORMATION PROPERTIES

The deformation modulus obtained from static and seismic tests in the field and in the laboratory can be compared. Based on the seismic down-hole tests in the field, the increase of the shear modulus with time after installation of LC columns can also be studied.

In order to compare axial strains (ε_a) from triaxial compression tests with shear strains (γ) from seismic tests, the following relationship is used

$$\Delta\gamma = (1+\nu)\,\Delta\varepsilon_a \qquad (12)$$

where ν is Poisson's ratio. At large strains (> 0.1 %) and undrained loading, it can be assumed that Poisson's ratio $\nu = 0.5$. Thus, $\gamma = 1.5\varepsilon_a$. As has been shown above, the main difference between the modulus determined from static or seismic tests is strain level, while the rate of loading is practically the same for both test types.

6.1 Effect of Time on Stiffness of LC Columns

The shear wave velocity was determined in-situ by down-hole tests at different time intervals after installation of two LC columns with different mixing quantities. In Figure 18, the increase of shear wave velocity with time (up to 41 days) is shown. The shear wave velocity of the clay before improvement was 40 m/s. Within a 41 days (approx. 1000 hrs), the shear wave velocity increased to 310 m/s. Also indicated are the values of shear wave velocity from the bender element tests (measured after 116 days). These tests suggest that the shear wave velocity continued to increase to about 360 m/s. The measurements give average values in the columns between 2.5 and 5.5 m depth.

No distinct difference could be observed between the shear wave velocities in the two columns with different mixing quantities (28 and 44 kg/m, respectively).

The maximum shear modulus, G_{max} can be readily calculated from the shear wave velocity. It was assumed that the density of the LC columns was 2.0 t/m^3, and that of the organic clay 1.4 t/m^3. Figure 19 shows the increase of the shear modulus with time in the LC columns, cf. Figure 18. The maximum shear modulus in the undisturbed clay was 2.3 MPa and increased after dry mixing within 41 days to about 190 MPa. The maximum shear modulus from bender element tests, performed after 116 days, reached an average value of 255 MPa. It can be assumed that the shear modulus will increase further with time. No significant difference in shear modulus could be observed between the two mixing quantities.

6.2 Effect of Strain on Stiffness of LC Columns

The shear modulus is strongly affected by strain level. In the case of a seismic test, the shear strain level is on the order of 0.001 %, or even lower. On the other hand, at conventional static triaxial tests, accurate measurements can be obtained at strain amplitudes higher than about 0.5 %. As has been shown in Figure 8, the modulus decreases significantly when shear strains increase from its maximum value to between 0.1 – 0.5 %. In Figure 20 the results of the triaxial tests are presented as a function of shear strain. Axial strain and elastic modulus were converted into equivalent shear strain and shear modulus values.

The shear modulus decreases with strain amplitude and increases with confining stress. The shear modulus (at 0.001 % shear strain: 255 MPa) decreased at 0.5 % to about 60 MPa. Thus, the shear modulus at relatively low shear strain level (0.5 %) is reduced to about 20 % of its maximum value. A strain level of 0.5 % is typical for the service state of most structures.

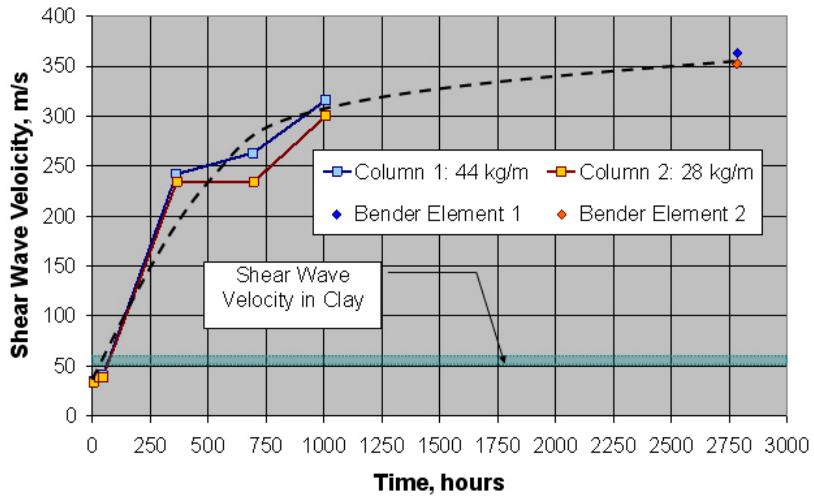


Figure 18. Variation of shear wave velocity with time, determined in-situ by down-hole tests and in the laboratory by bender element tests.

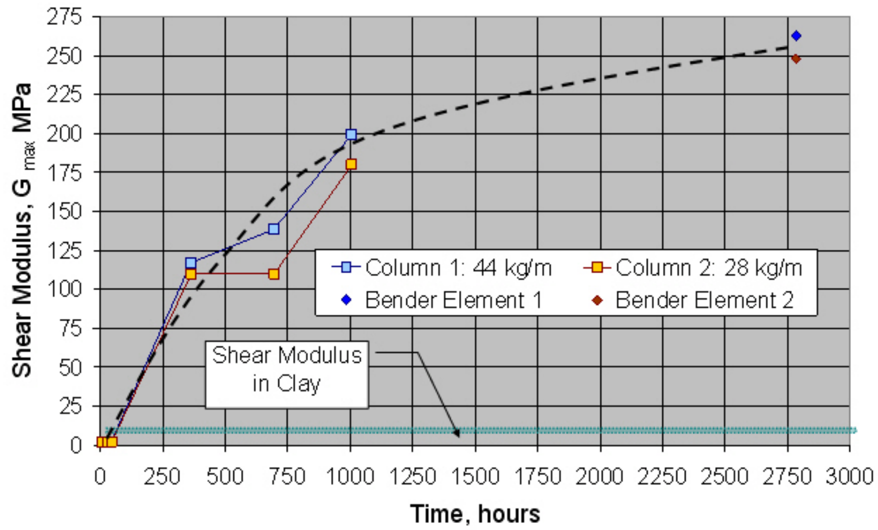


Figure 19. Variation of shear modulus G_{max} with time after installation of LC columns, cf. Figure 18.

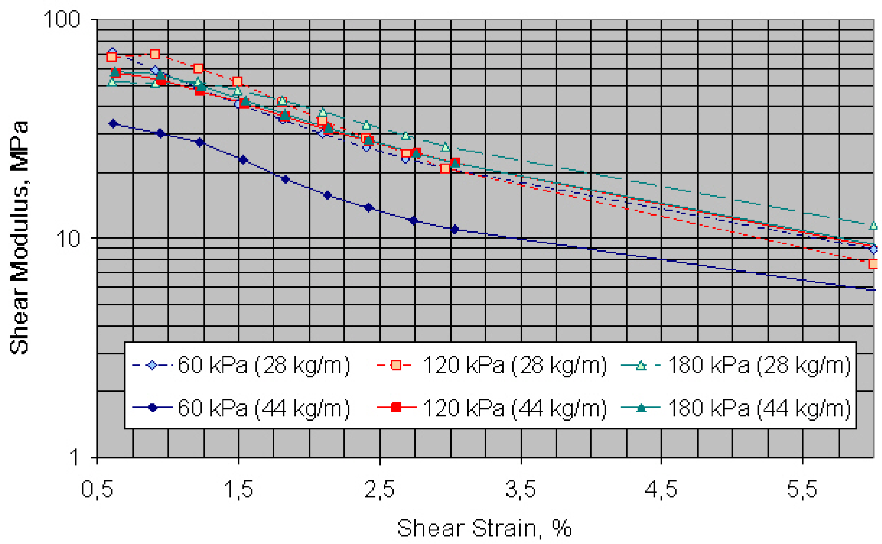


Figure 20. Shear modulus determined from triaxial tests within shear strain range of 0.5 to 6 %.

At failure (about 2 % shear strain), the average modulus is 25 MPa, thus only 10 % of the maximum value.

In Figure 21, the reduction of the shear modulus is shown as a function of shear strain for the results from triaxial and direct shear tests. The shear modulus has been normalized by its maximum value, determined by the bender element tests (255 MPa). The modulus reduction curves from the triaxial tests fall into a relatively narrow band. The shear modulus values from the direct shear tests are significantly lower. The lower modulus values in the case of the direct shear tests can be explained by the difference of failure mechanism (failure plane in a horizontal direction, which corresponds to the plane of mixing).

Figure 22 shows the shear modulus at approximately 0.6 % and 2.0 % shear strain, determined from triaxial tests at different confining pressures. The shear modulus is generally higher for the columns with lower mixing quantity (28 kg/m) – which is surprising, and increases with confining pressure. However, the increase is not as pronounced as could be expected from date in naturally deposited soils. This may be explained by the fact that the variation of the confining pressure for columns with high stiffness is relatively low.

6.3 Shear Strength of LC Columns

The shear strength of samples from LC columns with different mixing quantities was determined by undrained, triaxial and direct shear tests, Figure 23. An estimate of the average shear strength for the tested samples is given by the following relationship $\tau_f = 150\ kPa + \sigma \tan 45^0$.

The cohesion intercept is 150 kPa and the friction angle is 45 degrees. The lower boundary of all data is given by a cohesion intercept of 150 kPa and a friction angle of 30 degrees. Good agreement exists between the results from triaxial and direct shear tests. However, no distinct difference of shear strength could be observed between samples with low and high mixing quantities.

The ratio between the unconfined compressive strength and the elastic modulus can also be estimated, based on data shown in Table 2. The average value, E50/qu is 134 (97 – 174) for columns with a mixing quantity of 28 kg/m (99 kg/m^3) and 149 (76 – 252) for a mixing quantity of 44 kg/m (156 kg/m^3).

7 ESTIMATION OF DEFORMATION PROPERTIES BASED ON SEISMIC TESTS

The results presented in this paper demonstrate that it is possible to estimate the deformation properties of the unstabilized soil and of the LC columns using seismic methods. The first step is to estimate or measure the shear wave velocity. In fine-grained soils, Fig. 7 provides sufficiently accurate values of the shear modulus at small strain if the water content and the undrained shear strength are known. The shear modulus at operating conditions (shear strain range of 0.1 – 0.5 %, which corresponds to a factor of safety of 2 – 3), can be estimated using the modulus reduction factors shown in Fig. 8. For typical consolidated clays with a water content of 40 – 50 % and a plasticity index of 20 – 30, the shear modulus ratio at small strains, $G_{\max} / \sqrt{\tau_{fu}\, p_a}$ is about 900. Assuming a modulus reduction factor of R_m is about 0.3. In the case of clay with undrained shear strength of 10 kPa, the shear modulus at undrained loading is about 28 MPa. Assuming Poisson's ratio for undrained conditions: $\nu = 0.49$, the following modulus values are obtained: E = 84 MPa. The stiffness ratio E / τ_{fu} is then about 8500.

For the case of LC columns with a mixing ratio of 100 – 150 kg/m^3, the shear wave velocity after hardening will be on the order of 350 – 400 m/s, which corresponds to a shear modulus at small strains of $G_{\max}$ = 245 – 320 MPa. Results from laboratory tests, Fig. 21, indicate that the shear modulus decreases during undrained loading at 1 – 2 % shear strain to about 15% of the maximum value. Thus, the shear modulus at failure will be on the order of 37 – 48 MPa. The equivalent modulus values are E = 110 – 143 MPa. The undrained shear strength of the lime column depends on the confining stress, cf. Fig. 23. However, if an average value of 300 kPa is assumed, the ratio E / τ_f about 480. If a lower boundary value for the shear strength of 150 kPa is assumed, E / τ_f about 950. The equivalent values for the ratio E_{50} / q_u will be 240 – 475, cf. Fig. 9. However, the scatter in Fig. 9 is very large.

8 SUMMARY AND CONCLUSIONS

The static and dynamic modulus of soils is important for many geotechnical design problems. In spite of that, little guidance can be found in the geotechnical literature concerning the assessment of deformation properties of LC columns during static and dynamic loading.

Seismic tests can be used to determine the shear wave velocity in the field and in the laboratory. Different methods are available for evaluation of seismic test. The most reliable method for determining the shear wave velocity is by cross-correlation.

The shear modulus at small strains, G_{max} can be calculated based on shear wave velocity measurements.

The rate of loading during a seismic test is surprisingly slow and comparable to that of a conventional static test. Thus, it is possible to determine the static deformation modulus at small strains from seismic tests.

A semi-empirical relationship is proposed which can be used to estimate the shear modulus of fine-grained soils. The ratio of the shear modulus and the undrained shear strength can vary within a wide range, from 200 for plastic clays to 2000 for silty clays, cf. Figure 7. The most important parameter, which determines the relationship between the shear modulus at small strains and the undrained shear strength, is the natural water content (void ratio). For the analysis of many dynamic problems at the design stage, the empirical correlation given in Figure 7 (Eq. 7) may be sufficient. However, in organic soils, the underlying database is not sufficient to make reliable predictions.

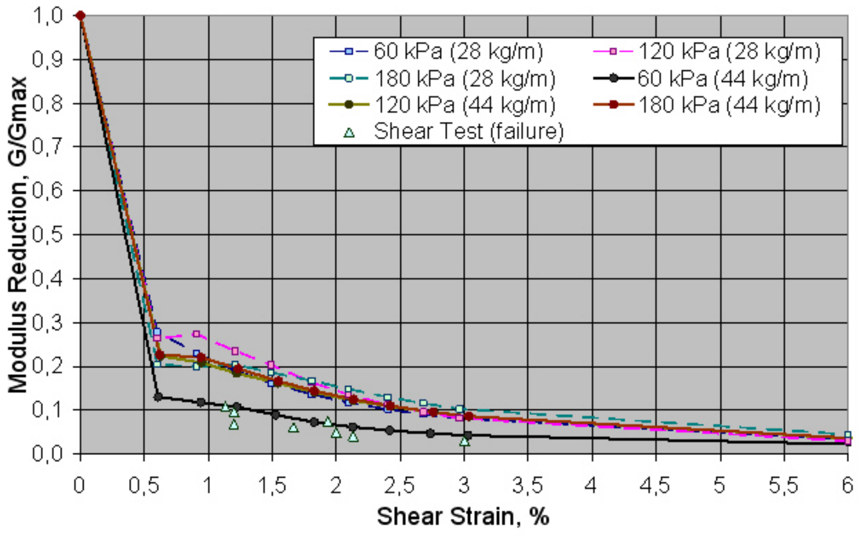


Figure 21 Decrease of shear modulus with shear strain for triaxial and direct shear tests.

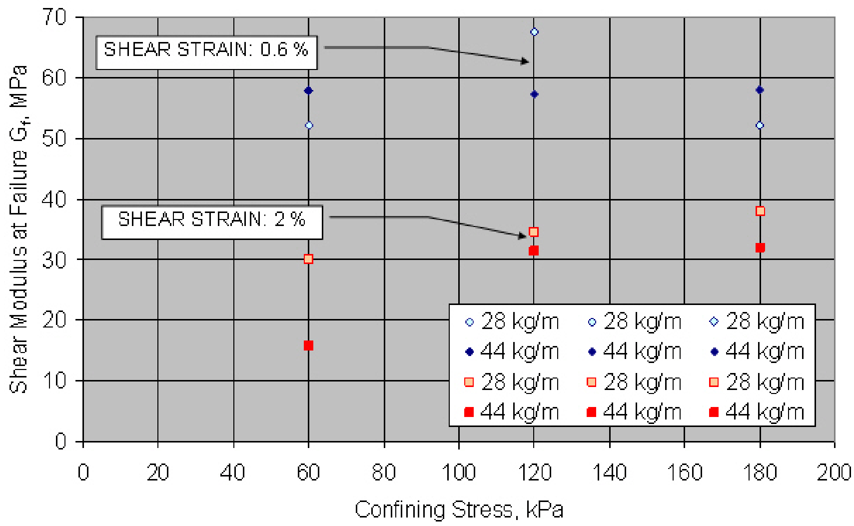


Figure 22. Variation of shear modulus G_{max} with confining stress at 0.6 and 2.0 % shear strain for columns with different mixing quantities.

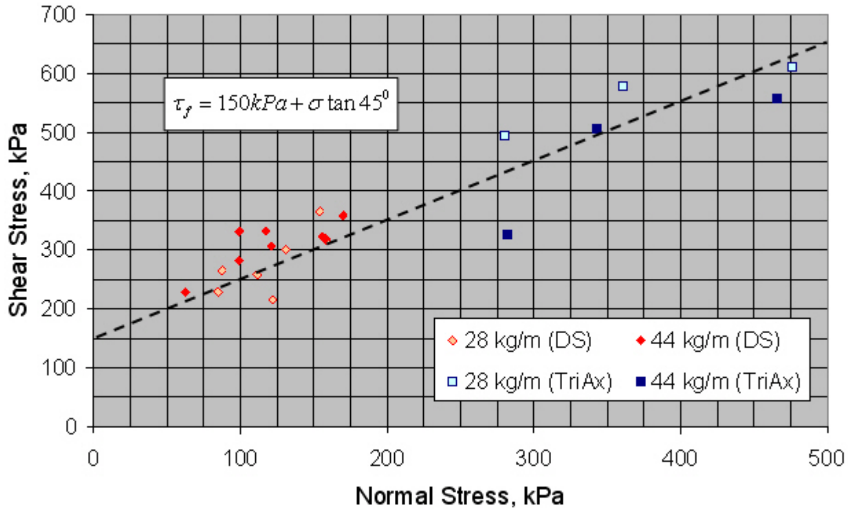


Figure 23. Shear strength of samples from LC columns determined by triaxial and direct shear tests.

It is generally recommended to verify the assumed modulus values by seismic field or laboratory tests.

The most important reason for the difference between the static and the seismic modulus is strain amplitude. Seismic tests are performed at very low shear strain levels (about 0.001 %). In the case of static tests the shear strain amplitude is in the range of 1 to 5 %. Within this strain range, the shear modulus decreases markedly and this effect can not be neglected.

Extensive field and laboratory tests were carried out on lime cement (LC) columns, manufactured by dry mixing, with two quantities of stabilizing agent (28 and 44 kg/m). Excavation of several columns showed that mixing produced relatively homogeneous columns. However, the content of mixing agent varied across the column area.

Seismic tests were performed in the field (down-hole test) and in the laboratory (bender element test). In the laboratory, triaxial and direct shear tests were performed on samples recovered from LC columns. The shear wave velocity in the soft, organic clay was about 40 m/s between 2.5 and 5.5 m depth.

The seismic tests clearly demonstrated that the shear modulus increases with time after installation of LC columns. Approximately 100 days after installation, the shear wave velocity increased in the columns to about 355 m/s. Based on limited data it can be assumed that the curing period is at least 100 days, but probably longer.

The maximum shear modulus of the LC columns was G_{max} = 255 MPa at 0.001 % shear strain. The shear modulus of LC columns appears not to be influenced significantly by confining stress.

The modulus was also measured by static triaxial tests. The shear modulus decreases at 0.6 % shear strain to about 20 % of the maximum modulus value, and at failure (2 %) to about 10 %. At normal operating conditions (at a strain level of approximately 0.5 %), the shear modulus is twice as high compared to failure conditions.

A method for estimating the stiffness of the unstabilized soil and of the stabilized lime columns is proposed, based on the results of seismic field and laboratory tests. The undrained shear strength of the investigated lime cement columns was determined by direct shear and triaxial tests. Good correlation between the two testing methods was obtained. The average shear strength can be defined by a cohesion intercept of 150 kPa and a friction angle of 45 degrees. The lower boundary is given by a cohesion intercept of 150 kPa and a friction angle of 30 degrees.

Surprisingly little difference of modulus and strength values was observed between samples with different mixing quantities: 28 (99 kg/m^3) and 44 kg/m (156 kg/m^3).

9 ACKNOWLEDGEMENTS

The investigations reported in the present paper were performed at the Royal Institute of Technology (KTH) and are based on two master thesis projects. The excellent work of Karin Axelsson, Johan Björkman and Johan Ryding made it possible to compare the comprehensive field and laboratory data. The thesis work was supervised by Mr. Johan Hagblom and Dr. Matti Kivelö.

Mr. Kent Allard developed the seismic field testing equipment and performed the field installation. The bender element tests were carried out at the Norwegian Geotechnical Institute under the supervision of Mr. Rune Dyvik.

The project was financially supported by the Swedish Railway Authorities (Banverket) and by the Swedish Development Fund of Building Contractors (SBUF).

Finally, the author wishes to acknowledge the valuable comments and suggestions made by the paper review committee.

10 REFERENCES

Andreasson, B. 1979. Deformation characteristics of soft, high-plastic clay under dynamic loading conditions. Doctoral Thesis. Chalmers Institute of Technology, 242 p.

Axelsson, K., 1996. Down-hole mätningar i kalk-cementpelare (Down-hole tests in lime-cement columns). Examensarbete 96/3, Department of Soil and Rock Mechanics, Royal Institute of Technology (KTH), Stockholm, Sweden, pp. 60.

Bjerrum, L. 1973. Problems in soil mechanics and construction on soft clays and structurally unstable soils. Proceedings 8th International Conference on Soil Mechanics and Foundation Engineering, Moscow, Volume 3 (State of the Art Report). Volume 3, pp. 111-160.

Björkman, J. and Ryding, J. 1996. Kalkcementpelares mekaniska egenskaper (Mechanical properties of limce-cement columns). Soil and Rock Mechanics, Royal Institute of Technology (KTH), Stockholm. Examensarbete 96/1, 58 + /43/ p.

Bodare, A. 1983. Dynamic screwplate for determination of soil modulus in situ. Doctoral Thesis, Uppsala University, UPTEC 83 79 R, 273 p.

Campanella, R. G., Baziw, E. J. and Sully, J. P. 1989. Interpretation of seismic cone data using digital filtering techniques. International conference on soil mechanics and foundation engineering, 12, Rio de Janeiro, August 1989. Proceedings, Vol. 1, pp. 195-198.

Döringer, H. 1997. Verformungseigenschaften von bindigen Böden bei kleinen Deformationen (Deformation properties of cohesive soils at small strain). Examensarbete 97/8, Department of Soil and Rock Mechanics, Royal Institute of Technology (KTH), Stockholm, Sweden, 54 p.

Dyvik, R. and Madshus, C. 1985. Lab measurements of Gmax using bender elements. Advances in the art of testing soils under cyclic conditions, Detroit, MI, Oct., 1985. Proceedings, pp. 186-196.

Ekström, J., 1994. Kontroll av kalkcementpelare (Control

of lime-cement columns). Slutrapport med redovisning av fältförsök i Ljungskile. Rapport B 1994:3. Göteborg, pp.154.
Hardin, B. 1978. The nature of stress strain behaviour of soils. Proceedings, ASCE Specialty Conference on Earthquake Engineering and Soil Dynamics, Pasadena, Vol. 1. pp. 3 – 30.
Kivelö, M., 1994. Odränerade provbelastningar av kalkcementpelare i fält (Undrained field load tests on lime-cement columns), Report 3002, Department of Soil and Rock Mechanics, Royal Institute of Technology (KTH), Stockholm, Sweden, 62 p.
Kivelö, M. 1998. Stabilization of embankments on soft soil with lime/cement columns. Doctoral thesis. Royal Institute of Technology. Division of Soil and Rock Mechanics. TRITA-AMI PHD 1023, 170 p.
Massarsch, K. R. 1979. Lateral earth pressure in normally consolidated clay. European conference on soil mechanics and foundation engineering, 7, Brighton, Sept. 1979. Proceedings, Vol. 2. pp. 245-249.
Massarsch, K. R., 2000. Application of Geophysical Methods for Geotechnical-, Geo Environmental and Geodynamic Applications - An Overview. Proceedings, 3rd International Workshop on the Application of Geophysics to Rock and Soil Engineering, 18. November, 2000, Melbourne, Proceedings pp. 1 - 5.
Massarsch, K. R. and Eriksson, H., 2002. "Deformation Properties of Dry Mixed-in-place Columns from Seismic Field Tests". Proceedings, Tokyo Workshop 2002 on Deep Mixing, Port and Airport Research Institute, Coastal Development Institute of Technology; pp. 58 - 74.
Massarsch, K. R. 2004. Deformation properties of fine-grained soils from seismic tests. Keynote lecture, International Conference on Site Characterization, ISC'2, 19 – 22 Sept. 2004, Porto, 133- 146.
Robertson, P. K., Campanella, R. G., Gillespie, D. and Rice, A. 1986. Seismic CPT to measure in situ shear wave velocity. ASCE. Journal of Geotechnical Engineering 1986, Vol. 112, nr 8, pp. 791-803.
Stokoe, K. H. and Santamarina, J. C. 2000. Seismic-wave-based testing in geotechnical engineering. Invited papers. Proceedings, GeoEng 2000. An international conference on geotechnical & geological engineering, Melbourne, Nov. 2000. Vol. 1. pp 1490-1536.
Stokoe, K. H., Sung-Ho J. and Woods, R. D. 2004. Some contributions of in-situ geophysical measurements to solving geotechnical engineering problems. Proceedings, 2nd International Conference on Site Characterization, Porto, Millpress Rotterdam, Vol. 1, pp. 97-132.
Vucetic, M. & Dobry, R. 1991. Effect of Soil Plasticity on Cyclic Response. Journal of the Geotechnical Engineering Division, ASCE Vol. 117, No. 1. Jan, pp. 89 - 107.
Woods, RD, Henke, R. 1981. Seismic techniques in the laboratory. ASCE. Geotechnical Engineering Division. Journal, Vol. 107, nr GT10, pp. 1309-1325.