State of Practice Report
– Execution, monitoring and quality control

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ABSTRACT: The state of practice of deep mixing concerning execution, monitoring and quality control is highlighted and discussed. Important recently published publications and different deep mixing methods are briefly reviewed. The mixing process in-situ is described and present knowledge concerning influencing factors are reviewed and discussed. The quality control concept of deep mixing is discussed and control methods are reviewed. The extent of testing and the evaluated strength- and deformation properties are discussed. The concept quality and variability in deep mixing are discussed.

1 INTRODUCTION
Ground improvement by deep mixing is today accepted world-wide in order to improve permeability and strength and deformation properties of soils. The experiences have been positive and deep mixing methods are undergoing rapid development, particularly with regard to the explicabilities and cost effectiveness.

The aim of the execution and installation process in deep mixing is to transport and distribute the binder in a way to produce a sufficiently uniform mixture. Compared to other mixing operations the execution and installation process in deep mixing is very complex. A series of operations and mixing mechanisms occur and it is difficult to monitor the entire process continuously in-situ. The wide variations in local geology, the difficulty of predicting the rheological properties of the soil-binder mixture, field conditions affecting equipment etc., all make it difficult to monitor, control and study the installation process in the field. Similarly, as for the installation process, the complex conditions result in special challenges with respect to the quality assurance of the stabilised soil. For example, it is still not possible to predict, from only tests on laboratory prepared specimens, the strength and deformation properties on a reasonable level of accuracy. There are many variables that differ between laboratory and field conditions related to the mixing process, curing conditions and environmental variables. The achieved properties in the improved soil must normally be estimated and verified by field tests and measurements, e.g. core sampling, soundings, in-situ tests and geophysical test methods.

In this report, the present state of practice concerning execution and quality control is outlined. Recently published knowledge is reviewed and discussed with a special focus on the papers presented at the conference. The mixing process in-situ and influencing factors are discussed. Different test methods are reviewed. The report also discusses the conception of quality, inherent properties variability, evaluated properties, and the extent of testing and sampling with special reference to statistical evaluations. Parts of the text is taken from the authors’ own work on the mixing process and stabilised soil variability (Larsson, 2003 and the corresponding papers Larsson, 2001; Larsson et al., 2005a,b,c).

2 IMPORTANT PUBLICATIONS
A large number of general reviews concerning deep mixing have been presented during the years, primarily in connection to conferences. In spite of a great interest and a large amount of publications, few deal with the execution of deep mixing, especially the mixing process in-situ and influencing factors. Somewhat more papers deal with quality assessment and test methods.

The IS-Tokyo´96 conference (Yonekura et al., 1996) is a benchmark in an international perspective. During the 1980s, deep mixing methods developed explosively in Japan. During that time, the Japanese technologies and experiences were presented mainly in Japanese journals and in national conferences. However, at the IS-Tokyo´96 conference a wide range of Japanese technologies were presented in a large number of papers.

Dry deep mixing in Sweden was developed in the mid 1970s by principally one contractor. During the 1980s, the development of dry deep mixing was mainly provided by government clients, research institutes and universities. The tardy development concerning execution and the installation process was mainly due to a limited market. An extensive and rapid development started however in connection to a large investment program for infrastructure projects at the end of the 1980s. The “Dry Mix Methods for Deep Stabilization” conference in Stockholm 1999 (Bredenberg et al., 1999) and the GIGS conference in Helsinki 2000 (Rathmaier, 2000) provide surveys of dry deep mixing in the Scandinavian countries.
A workshop on deep mixing was held in Tokyo 2002, organized by the port and Airport Research Institute and the Coastal Development Institute of Technology in collaboration with CEN TC288/WG10. The proceedings from this workshop provide an overview of current practice and recent trends especially in Japan (Kitazume & Terashi, 2002).

In the beginning of the 1990s, deep mixing methods were used in increased proportions in Europe, Asia and North America. The number of papers concerning deep mixing also increased dramatically. Not only because of the increased number of English-speaking authors, but also because Japanese and Scandinavian operators started to write papers in English. However, there are still few papers concerning execution and quality control respectively.

A number of “review” papers recently presented at conferences and workshops (e.g. Hosoya et al., 1997; Yoshizawa et al., 1997; Bruce et al., 1998a, 1999, 2000; Halkola, 1999; Bredenberg, 1999; Holm, 2000; Terashi & Juran, 2000; Terashi, 2002b, 2003; Usui, 2005), the author recommends the journal papers by Porbaha et al. (2001a) and Porbaha (2000) concerning execution and quality control respectively.

A survey on deep mixing methods is presented by Bruce (2000) and an extensive and excellent survey of deep mixing methods is presently presented by Topolnicki (2004). Furthermore, the Japanese guideline, CDIT (2002), deals with Japanese techniques and experiences. A very important document in the near future is the European standard concerning execution, prEN 14679 (2005), which is shortly presented by Hansbo & Massarsch (2005). Other publications and conferences/seminars of interest are listed in three Regional reports of Europe (Massarsch & Topolnicki, 2005), Asia (Nozu, 2005) and Northern America (Porbaha et al., 2005).

3 EXECUTION

3.1 Introduction

The principal purpose of execution and the mixing process in deep mixing is to bring about changes in physical properties in a soil. Mixing takes place between two materials when they come into contact with each other and are deformed so that the contact area between them increases. Soft soils like clay have high resistance to deformation due to viscous elastic properties, i.e. movements only take place in a narrow volume around the mixing devise. Relatively large forces are necessary to overcome this resistance. Mixing also takes place when materials are cut into smaller elements and redistributed. A satisfactory mixing process produces a mixture of the required uniformity in the shortest possible time and at the lowest possible cost in terms of machinery, power and labour.

There are a number of newly presented reports concerning execution and monitoring, especially focusing on different deep mixing methods, the machinery and technical data. The author considers these reports to be extensive, well written and highly recommended, and therefore, this report focuses on the mixing process in-situ and not on the machinery. Furthermore, European standard concerning execution prEN 14679 (2005) provides a checklist for the information needed for the execution of the work, geotechnical investigations etc.

The European standard prEN 14679 (2005) frequently uses the term “mixing” defined as “the mixing process involves mechanical disaggregating of the soil structure, dispersion of binders and fillers in the soil”. In spite of the fact that the term “mixing” is a central conception, few publications really treat the term in a stringent and scientific sense.

3.2 Deep mixing methods

From a scientific point of view, it is difficult to separate shallow soil stabilisation and deep mixing. According to prEN 14679 (2005) deep mixing involves “mixing by rotating mechanical mixing tools where the lateral support provided to the surrounding soil is not removed”. Traditionally, shallow soil stabilisation is related to the shallow subgrade under e.g. road constructions, whereas deep mixing is related to the improvement of the whole soil deposit. According to prEN 14679 (2005) deep mixing involves “treatment of the soil to a minimum depth of 3 m”. Most of the deep mixing methods uses paddle formed blades or augers mounted on one ore more shafts.

During the 1980s a number of different deep mixing methods were developed in Japan, most of them having unique names. Due to the large number of techniques a classification of deep mixing methods was necessary when the use of deep mixing increased world-wide. A rigorous classification is proposed by Bruce et al. (1998a,b and 1999). Fig. 3.1 shows a general classification of deep mixing methods according to Topolnicki (2004).
The classification is based on (a) binder form, (b) mixing principle and (c) location of mixing action.

**Wet mixing**
The Japanese Cement deep mixing (CDM), or the “wet method” of deep mixing method (CDIT, 2002), was developed in the mid 1970s and represents the largest group of wet mixing techniques. In 1977 the CDM Association was set up to coordinate the development of the method in collaboration between industry and research institutes. Deep mixing by the “wet method” has been extensively used in Japan, particularly in marine projects. The development of deep mixing machines and a description of the execution procedure of the Japanese wet method are presently presented by Terashi (2002b) and Nakanishi (2002) respectively.

There are a variety of ways of injecting water-based binders into the soil. A common approach is to inject part of the binder as the mixing tool is penetrating the soil, e.g. from the tip of a screw or paddle. As the mixing tool penetrates,
it disaggregates the soil and at the same time lifts it slightly in order to facilitate incorporation. The remainder of the binder is injected as the tool is withdrawn from the soil. On land, usually one or two mixing shafts are used and the diameter of the columns is about 1 m (Nakanishi, 2002). Fig. 3.2a shows the CDMLand4, where four shafts are combined. By providing the tool with an outer and an inner rod rotating in opposite directions it is possible to mix frictional materials by a mixing tool 2 m in diameter (Isobe, 1996).

It is common in Japan, especially in marine works, to install a number of columns simultaneously, using multiple shafts and mixing tools, Fig. 3.2b. It is possible by this method to efficiently construct configurations such as blocks, lattices, walls etc. (e.g. Kawasaki et al., 1981; Nicholson et al., 1998). Column walls can be reinforced with I-beams (Schefer et al., 1997). Reinforced columns can be used to construct gravity walls, VERT (Vertically earth reinforced technology), (Nicholson et al., 1998; Andromolos et al., 2000).

The size of the mixing tools and the machines are gradually increased in order to increase the construction speed (Terashi, 2002). The mixing tool of the recently developed CDM-Column21 (Yoshida, 2002) method is equipped with a large number of blades in a rather complex design, Fig. 3.2c. Recently developed equipment uses four shafts for on-land works, Fig. 3.2a (Yoshida, 2002; Yoshida & Kawashima, 2005). A problem that may occur when blades are lying closely together is that the soil-binder mixture is clogging between the blades. The mixing tool can be provided with "anti-rotation vanes" which prevent the materials being mixed from rotating along with the tool. These vanes do not rotate in the soil and thus provide resistance to the rotating blades as they pass. These vanes are somewhat longer than the rotating blades and thus cut into the surrounding soil for support. The technique was presented in the early 1980s by e.g. Inoue & Hibino (1985) and Enami et al. (1986).

CDM-LODIC (Cement Deep Mixing – Low Displacement and Control) was developed in 1985 in order to minimize ground movements (Sugiyama, 2002; Tanaka et al., 2002; Matsumoto et al., 1998). The principal is to remove a volume of soil equivalent to the volume of the mixing tool and the incorporated cement slurry. The whole lengths of the shafts are therefore equipped with a screw as shown in Fig. 3.2d. This technique makes the installation of large diameter columns (1.2 to 1.3 m) possible without large ground movements (Kamimura et al., 2005). A similar technique, Soil Removal Technique, is also presented by Hirai et al. (1996).

Wet deep mixing was developed in the U.S. in the mid 1980s by Geo-Con Inc. and SMW Seiko Kogyo at the Jackson Lake Dam (Ryan, 2005). The mixing tools were originally similar as the Japanese techniques but a number of companies in the U.S. have developed both single and multiple axis mixing devices as shown in Fig. 3.2. A description of mixing tools, machinery and the execution procedure used by Hayward Baker is presently presented by Burke (2002).

Deep mixing soil equipment has been developed in countries in Europe, such as Germany, France, England, Italy and Poland (Stockar & Seidel, 2005; Harnan, 1993; Harnan & Iagolnitzer, 1994; Paviani & Pagotto, 1991; Pagliacci & Pagotto, 1994; Topolnicki, 2002, 2003). A number of mixing tools that have been used in Europe are shown in Figs. 3.4a-d. The Colmix method, developed by Bachy in the late 1980s, involves mixing the soil with a water-based or dry binder by means of a helical tool. The binder is injected as the tool penetrates the soil. Mixing and compaction take place as the tool is withdrawn. The Trevimix method was developed in Italy in the early 1980s and use both dry and wet binders. Keller and May Gurney have been used wet deep mixing since the end of the 1990s.

In Japan it has also been a common practice to install rectangular columns (Mizutani et al., 1996; Watanabe et al., 1996). Technique for creating rectangular columns was developed in the early 1980s (Khono 1984). The main objective of the use of rectangular columns is to avoid overlap between circular columns. Two examples of equipments for this purpose appear in Figs. 3.5a and b. A somewhat similar technique, Cutter Soil Mixing, was recently developed in Europe (Fiorotto et al., 2005) as shown in Fig. 3.5c. Two vertically mounted cutter units create rectangular panels. The length of the panels varies between 2.2 and 2.8 m and the width varies between 0.5 and 1.0 m. The panels can be combined in order to form different configurations such as rows, grids, blocks etc.

In order to isolate waste disposal sites and to prevent the movement of toxide substances, deep mixing methods can be used. It is common to create column walls, grids or blocks in the ground. A relatively newly developed soil mixing wall method, TRD method (Trench cutting Remixing Deep wall method, Kamon, 2000), is conducted with a chain-saw cutter device as shown in Fig. 3.5d. The advantage, compared to walls created by columns, is the secured continuity of the wall. By motion of the chain the slurry around the cutters forms eddies and turbulent mixing occurs. A similar technique is developed in Germany called the Cut-Mix-Injection method (Sarhan, 1999), Fig. 3.5e. A 1 m thick rigid wall is created by a saw-shaped cut and mixing tool.

As shown there exist a large number of deep mixing methods developed in Japan. Special-designed equipment has been used to improve the soil in horizontal direction under a railway embankment in order to improve the soil as temporary support, HEMS (Sugiki & Meada, 1996). Another system is a combined horizontal nail and cement deep mixing, RADISH (Tateyama et al., 1996). The principal is to increase the pull out capacity for the anchoring of e.g. gravity walls.
Figure 3.2 Wet deep mixing methods in Japan

(a) CDM Land4
(courtesy of CDM Association)

(b) CDM marine. DCM-2 provided with eight-shaft mixing tool (Terashi, 2005)
(courtesy of CDM Association)

(c) CDM-Column 21
(courtesy of CDM Association)

(d) CDM-Lodic
(courtesy of CDM Association)
Figure 3.3 Wet deep mixing methods in the U.S.
Figure 3.4 Wet deep mixing methods in Europe

(a) Colmix (courtesy of Bachy Soletanche Ltd) (a) May Gurney (courtesy of May Gurney)

(c) Keller, Poland and UK respectively (courtesy of Keller)

(d) Three mixing tools for different soil conditions used by Trevi (courtesy of Trevi)
Figure 3.5 Wet deep mixing by vertical rotating mixing tools
Dry mixing

The two major dry method techniques are the Japanese DJM and the lime-cement column method, today referred as Japanese and Nordic technique, respectively, according to prEN 14679 (2005).

The Public Works Research Institute of the Japanese Ministry of Construction developed the "Dry jet mixing (DJM) method" that is the largest group of dry mixing techniques. According to CDIT (2002) about the same volume of stabilised soil is carried out by dry and wet mixing, respectively. In the early 1980s a number of tests were performed in order to develop mixing tools (Nishibayashi et al., 1984). A variety of basic designs of mixing tool and different ways of incorporating binders into the soil were tested at this time. One of these was the method of allowing the binder to spread over the cross section of the column in the cavity formed behind the paddle of a mixing tool. Fig. 3.6a shows the standard mixing tool by the Japanese "dry method", based on the principle that the binder is spread over the column cross section in the cavity formed behind the blades of the tool as it rotates in the soil. Faster rotation speeds increase the volume of the cavity and create a vacuum, which facilitates the process when manufacturing large-diameter columns (Chida, 1982). Further, according to Chida (1982), this process causes the injected air to move towards the outer parts of the cavity. The air is then evacuated from the column via the shaft of the mixing tool. A mixing tool diameter of 1m and double shafts are adopted as standard in Japan (Yasui et al., 2005). Equipments for the installation of columns of 1,3 m in diameter are also available. Descriptions of the execution procedure of the Japanese dry method (DJM) are recently presented by Aoi (2002) and Yasui et al. (2005). Takeda & Hioki (2005) present a development of the Japanese dry deep mixing tool where the binder is incorporated from the end of the mixing blade. The binder outlet hole is placed on the edge of the mixing blade and by which the air-binder mixture is injected towards the mixing shaft.

The Nordic dry deep mixing method was set into practice in the mid 1970s. Development of equipment for dry mixing in Sweden was begun in the early 1970s by Lindén-Alimak AB. The method was adapted for the market by BPA Byggnadsproduktion AB (Boman & Broms, 1975). Research and development on dry deep mixing started in Finland at the same time (Rathmayer, 1997). In the years 1975 – 1979 many types of mixing tool were tested (Wikström, 1979). The aim in the early stages of development was a device of high production capacity. In coarser and more solid soils an auger type tool gave good results. A mixing tool similar as the tool shown in Fig. 3.6b was found to give the best results in soft clay. A tool with a number of tilted vanes (paddles) was tried, but it was found that the clay stuck to the tool, impeding mixing. The trials with the early tools are unfortunately unpublished and it is therefore not possible to review and comment on the results.

Since the initial development of mixing tools in the 1970s, most projects have been carried out with tools of the type in Figs. 3.6c and 3.4d. Slight variations of the "standard tool" exist as shown in Fig. 3.6c. Further development has been very limited. On the machinery side, however, development has been dramatic. Description of the execution procedure of the Nordic dry method is recently presented by Bredenberg (1999) and Larsson (2003).

A development of the Nordic dry deep mixing method is introduced recently by LCTechnology (2002), where some problems in connection to relatively dry and hard soils are overcome by adding water separately during insertion of the mixing tool, Fig. 3.6e. The soil profile can thereby obtain consistent water content. The added water can also act as a lubricant for the mixing tool during penetration. A similar technique has been reported by Wiggers & Perzon (2005).

Shallow soil mixing

Shallow Soil Mixing (Broomhead & Jasperse, 1992; Aldridge & Naguib 1992; Day & Ryan, 1995) is a technique developed by Geo-Con in North America in the early 1990s to improve large mass of soils within ten meters depth below the surface. The process uses a single mixing tool 1 m to 4 m in diameter as shown in Fig. 3.7a. Slurry grout is incorporated into the soil at the bottom of the mixing tool via three ports, located in the auger flights. Dry binder is used and distributed pneumatically if sludges or wet soils are stabilised. A common application for deep mixing in US is environmental cleanups, contain, stabilise or treat the soils to permit safe closure. Fig. 3.7b shows a modern version of a mixing tool used by Geo-Solutions Inc.

Several shallow soil mixing techniques has been used in Japan (Terashi, 2002). Two techniques are shown in Fig. 3.7c. On the left, the mixing machine is located on a float that is dragged on an extremely soft soil by a winch. On the left, a technique similar as the Finnish mass stabilisation is shown. However, the Japanese technique was introduced in the late 1970s.

In the early 1990s, mass stabilisation was developed in Finland for the stabilisation of peat or gytta (Hoikkala et al., 1996). The dispersion of binders is carried out by a mixing tool mounted on an excavator according to Fig. 3.7d. The soil is stabilised to 5 m depth by this technique. This equipment makes the process simple and flexible. Within a few hours of mixing, 1 to 1.5 m of fill is applied. Another mixing tool developed in 2001 is shown in Fig. 3.7e. The drum rotates by approximately 200 rpm and the soil is disaggregated before the binder is added. This large rotary cultivator puts a relatively large volume in motion during mixing. The stabilisation of peat or gytta can also be performed by installing columns in blocks (e.g. Hansson et al., 2001; Dahlström & Eriksson, 2005).
Figure 3.6 Dry deep mixing methods

(a) DDM standard (courtesy of DDM Association)
(c) Three versions of the Nordic dry mixing “standard” tool (courtesy of Hercules Grundläggning and LCM)
(d) Nordic dry mixing “Pinnbör" (courtesy of LCM)
(e) Modified dry deep mixing (LCTechnology, 2002)
(a) Shallow soil mixing (U.S.)
(Day & Ryan, 1995)

(b) Soil mixing (U.S.)
(courtesy of Geo-Solutions Inc.)

(c) Shallow soil mixing (Japan)
(Japan Cement Association, 1994)

(d) Mass stabilisation where the mixing tool is mounted on an excavator
(Hoikkala et al., 1996; Axelsson & Säfström, 1996)

(e) Mass stabilisation where a large rotary cultivator is used
(Niska & Nyysönen; courtesy of ALLU, Finland)

Figure 3.7 Shallow soil mixing methods
Combined jet and wet deep mixing

Fig. 3.8 illustrates methods, which are combinations of deep mixing using mechanical mixing and jet mixing. The main advantage of these methods is that they can produce large-diameter columns without large, bulky mixing equipment. The oldest of the methods is SWING (Spreadable Wing Method), developed at the beginning of the 1980s (Kawasaki et al., 1996; Yang et al., 1998; Ogawa 1990). Similar methods have subsequently been developed incorporating evolved forms of jet mixing: LDis, (Ueki et al., 1996); JACSMAN (Miyoshi & Hirayama, 1996; Mori et al., 1997; Matsumoto et al., 1998; Kawanabe & Nozu, 2002)

Also in USA a combined technique between jet mixing and mechanical mixing is developed, Geo-Jet (Reavis & Freyaldenhoven, 1994; Craft, 2004). The mixing tool is provided by two relatively broad paddles. When the mixing tool is rotated into the soil, cement slurry under high pressure is discharged via a number of nozzles placed along the blades. The combination of mechanical mixing and hydraulic mixing creates a liquefied mixture of soil, and cement.

Mixing above ground surface

In Japan there are methods using mixing plants on the ground surface (Mori et al., 1996). A continuous auger is used to mix the soil and the binder. In an initial step, soil is transported to the ground surface as the rotating helical mixer is inserted into the earth. The soil is mixed in a mixing plant with a water-based binder and then pumped back into the ground as the tool is withdrawn. This method yields a controllable product, comparable to concrete, whose strength and deformation properties can be varied. The displacement caused by the injection of binder in-situ is reduced. The construction of man-made islands in Japan has resulted in a development of methods for the reclamation of dredged soils. In order to stabilise large quantities of dredged soil a pneumatic flow mixing method has been developed (Kitazume & Satoh, 2003; Hayano & Kitazume, 2005). The principal mixing equipment is a static mixer where the dredged soil is transported through pipe by means of compressed air. The soil-binder mixture forms separated mud-plugs in the pipe, which are mixed by turbulent flow generated in the plug. The properties in the stabilised soil are predictable and the control is relatively easy to perform. The mixing plants are, however, enormous and built for large projects and large quantities.

The principle to mix above ground surface has also been used in Finland in connection with the building of a new seaport in Vuosaari (Lahtinen et al., 2005). Excavated poor-quality soils, such as silt, are stabilised with a stack mixer. The stabilised soil is used for the construction of embankments, field structures, noise barriers etc.

(a) SWING
(courtesy of SWING Association)

(b) JACKSMAN
(courtesy of CDM Association)

(c) LDis (Ueki et al., 1996)
(courtesy of Onoda Chemical Co Ltd)

(d) Geo-Jet
(courtesy of Condon-Johnson & Associates Inc.)

Figure 3.8 Combined jet and wet deep mixing methods
3.3 Monitoring during execution

Normally the whole machinery process is fully automated and controlled by computer systems. There are sensors and gauges to measure the amount of incorporated slurry or dry binder, depth, rotation, speed and torque. The torque is however normally not measured in Sweden. In the Scandinavian countries the monitoring normally includes the amount of binder, retrieval rate and rotation speed.

3.4 The mixing process

The purpose of the mixing process is to disperse the binder in the soil so as to provide the best possible conditions for the chemical reactions to take place. Mixing is in this report defined as an operation which tends to reduce no uniformities or gradients in composition and properties in a soil-binder mixture. If all of the binder is to contribute actively to the improvement of the soil, the particles of binder must all be evenly dispersed throughout the volume of the column. Moreover, the binder should be evenly distributed over the column cross section in order to limit the variability of strength and deformation properties. Wide dispersion of the properties not only reduces the predictive value of laboratory tests but also complicates production control. The consequence may be a loss of control of the process and its results.

The mixing process in deep mixing is very complex, comprising many phases, and many factors influence the process and its result. It is difficult to clearly separate the different mechanisms involved in the mixing process, however it is important to understand how the mechanisms affect each other. The installation process in dry deep mixing as it is executed in the Scandinavian countries can be divided into three principal phases:

1) penetration of mixing tool to the required depth;
2) dispersion of binder;
3) molecular diffusion.

Differences and similarities between this division of the mixing process and the mixing process for the wet mixing method are briefly discussed in section 4.6.

Penetration of mixing tool

In the first phase of the mixing process the rotating mixing tool is driven into the soil to the desired depth. The insertion process can be executed in such a manner that the resulting remoulding and disaggregation of the soil changes the conditions for subsequent phases. Total disaggregation (dispersed and deflocculated structure) can have a positive effect on the active mixing mechanisms by making it easier to produce the necessary movements in the soil for the mixing of the materials. However, existing mixing tools in the Scandinavian countries are equipped with paddles which are generally set at a small angle to the horizontal. This facilitates insertion of the tool, as it requires little energy to penetrate the soil. The inevitable consequence, however, is that relatively little energy is expended on remoulding the soil. It takes movement of the soil to produce the shear forces necessary for disaggregation.

An important issue is whether increasing the agitation energy during penetration of the tool actually improves conditions for the mixing process sufficiently to justify the extra cost it entails. It is uncertain today how much the result of mixing is affected by the input of agitation energy during penetration as compared with other factors. The tool is currently inserted at the rate of approximately 100 mm/rev, and it is doubtful whether the soil structure is significantly affected. It is also doubtful whether it is economical to remould and disaggregate the soil before adding the binding agent.

Furthermore, it is not certain that complete disaggregation of the soil will have exclusively positive effects on the strength gain. Complete disaggregation combined with poor mixing efficiency (high concentration variances) may result in low shear strength of relatively massive striations and/or lumps impairing the overall strength properties of the column.

In the current mixing procedure, there is a risk that much of the energy used in the dispersion process is expended on remoulding the clay. Laminar mixing is likely to be less effective as the clay can withstand deformation thanks to elastic drag and high yield point.

The dispersion process

The process by which the binder is dispersed in the soil can be divided into four steps (basically based on a division of the dispersion process by Parfit & Barnes (1992)):

a) incorporation and spreading of the binder;
b) wetting of solid particles;
c) breakdown of agglomerates;
d) distribution.

In practice the stages overlap and it can therefore be difficult to distinguish them by visual observation.

a) Incorporation and spreading of the binder

It is most important that the mixing tool is designed in such a way that the binder is spread as evenly as possible over the column cross-section during the incorporation and spreading phase. This avoids major concentration variances, large agglomerates (loosely bound lumps), makes the breakdown of agglomerates easier, and avoids long mixing times, since the whole dispersion process takes place during a short time of mixing.

In the Scandinavian countries, binder is delivered in powder form from tanks via hoses to the mixing tool using compressed air as a transport medium. The binder is expelled through a hole in the Kelly rod in connection to the upper par of mixing blades, usually in the manner shown in Fig. 3.6c. The binder is spread through the
cavities formed in the soil by the rotating mixing tool. The size and shape of the cavities formed depends on factors such as the geometry of the mixing tool and the outlet hole, the intensity of mixing, the air pressure, stress conditions in the soil, the rheological properties of the soil and the binder.

b) Wetting of solid particles
When a powdered binding agent is mixed with a soil, the lumps of powder contain entrapped air which must be replaced with liquid. In e.g. calcareous and cementitious reactions, liquid is drawn from the soil by diffusion. If the air is not to remain entrapped, mechanical work has to be done, but even with mechanical assistance the release of the air can be difficult if agglomerates or aggregates have formed. The wetting process is assisted by mixing tools, which generate high compressive and shear stresses in the soil.

The wetting process cannot occur spontaneously because the lime and cement particles are not dense enough to sink into the clay slurry. The high particle concentration, the complex spreading process, the presence of air as a component, and the chemical reactions proceeding during the mixing process make the behaviour of the mixture extremely complex. The wetting process is influenced by the properties of the liquid phase, the character of the surface, the dimensions of the interstices in the agglomerates, and the compressive forces exerted by the mechanical system on the components. The rheological behaviour of the clay (dependent on e.g. clay and water content) is also highly significant for the wetting process. Complete remoulding of the clay releases water which is available for wetting the lime and cement particles. It also gives the clay particles a larger active interfacial area with the binder, which promotes molecular diffusion.

c) Breakdown of agglomerates
Once the binder is incorporated and spread and the particles are wetted, the agglomerates that have formed should break down before the chemical reactions begin. Unless the particles are evenly dispersed in the mixture, large aggregates will form, resulting in concentration variances and poor mixture quality. Furthermore, if the particles are not dispersed, not all the particles of binder will make their full contribution to even strength gain throughout the stabilized volume.

Agglomerates are broken down by shearing or by large compressive forces. To produce sufficiently large shearing or compressive forces the mixture must be set into motion. For the effective breakdown of agglomerates in a mixture with dough-like properties, experience from process industries indicates that equipment with a kneading or grinding action should be used. The mechanical mixing is generally done by rotating impellers of paddle or helical type. Paddles and screws are often combined. A kneading action in the mixture may be produced by paddles mounted at a relatively large angle to the horizontal, thus generating both axial and tangential motions. It is advantageous to generate movements in a number of directions because clayey soils have visco-elastic properties, entailing a risk that the mixing tool will only generate movement in the immediate vicinity of the blades, i.e. that the soil will "yield" in a thin shear plane close to the tool due to the fact that the intermediate forces around the impeller rapidly die out. The impeller may not generate significant movement of the mixture, with the result that if the mixing time is too short, layers of solid additive will be left in the soil. Hence, it is important to design the mixing tool to produce forced movement of the materials and generate laminar mixing within a sufficient volume of soil around the tool.

Depending on the rheological properties of the soil and on which mixing mechanisms are active, the efficiency of the mixing process is influenced mainly by the following two factors:

1) the effective strain in the mixture. This factor can be expressed as a function of the number of revolutions per metre (rev/m) or the retrieval rate (mm/rev) of the mixing tool;

2) the intensity of mixing or agitation. This factor can be expressed as a function of the rotation speed of the mixing tool (rev/min).

A hypothesis in this connection is that when the mixing intensity is sufficient to break down agglomerates, the effective strain is the most significant factor for better mixture quality. If on the other hand the mixing intensity is too low to break down the agglomerates, the effective strain is of little importance as the agglomerates are merely entrained with the mixture. According to this hypothesis, it is the rheology of the soil that determines how far the retrieval rate and rotation speed of the tool influence the process.

If the soil is completely remoulded and behaves like a liquid, under favourable conditions, turbulent flow may be generated in the mixture. Turbulent flows result in effective circulation and high shear forces. If the viscous drag in the fluid is small, a larger proportion of the mixing energy will go into breaking down agglomerates. It takes relatively high rotation speeds to produce turbulent flow. At low speeds there is a risk that the tool will merely move the agglomerates around without breaking them up.

d) Distribution
Distribution is the process by which the disaggregated agglomerates are randomly scattered through the mixture. This normally takes place concurrently with the preceding process.

If the binder has not been adequately spread during the previous phases of the process, long mixing times may be expected, as it is difficult to generate movement in all soil types. Distribution of the binder is probably promoted by complete disaggregation of the soil, high water content, and
low viscosity. It will only be successful with mixing tools that produce large movements in the soil.

**Molecular diffusion**

After the execution, the mixing process continues by molecular diffusion, mainly by the migration of calcium ions from the stabilised soil into the unstabilised surrounding soil, or from regions of the stabilised soil with a high concentration of calcium ions into parts with lower concentration. The migration of calcium ions has been a subject for relatively many investigations (e.g. Rogers et al., 2000a,b; Rogers & Glendinning, 1994, 1997; Rajasekaran & Narasimha Rao, 1997, 2000; Hayashi et al., 2003; Larsson & Kosche, 2005). Calcium ions migrate approximately 30mm within roughly a year (Rogers & Glendinning, 1996) and about 50mm in 10 years (Löfroth, 2005). The binder may also migrate into shrinkage cracks in the surrounding (Rao & Thyagaraj, 2003) and into vertical fractures caused by lateral pressure from the injection of binder, and by the shearing of the soil caused by the mixing tool (Shen & Miura, 1999; Shen et al., 2003a,b). The installation process may cause clay fracturing in the range of 2-3 times the column diameter.

However, these investigations focus on the migration from the stabilised soil into the unstabilised surrounding soil. The process where calcium ions migrate from regions of the stabilised soil with a high concentration of calcium ions into parts with lower concentration is unfortunately not investigated. It is therefore difficult to evaluate the extent of this process and state whether this process is significant or not for the mixing process. However, observations in extracted lime-cement columns show that the columns seem to heal, a short time after column penetration tests, possibly due to migration of calcium ions (Axelsson & Larsson, 2003a,b). It was difficult to determine the placing of the probe at the visual examination.

A common opinion is that it requires a higher amount of mixing using cement as binder compared to lime. This statement is often based on the argument that the intensity of migration of calcium ions is less in cement treated soil. Without questioning this argument, it must be emphasized that there are no tests published validating this hypothesis.

### 3.5 Factors affecting the mixing process

The mixing process in dry deep mixing is very complex. A variety of factors may influence the process and its results:

- the geometry of the mixing tool;
- the mixing energy: the retrieval rate and rotation speed of the mixing tool;
- the consolidation stress, the compaction energy, the temperature, the availability of water and seepage flow, which affect molecular diffusion.

This section reports and discusses published studies on factors influencing the mixing process. An overview of a number of investigations reported in relation to deep mixing is shown in Table 1. As shown in the table, the focus has been on the influence of the mixing work and on comparisons of different mixing tools. The influence of factors such as the rotation speed, air pressure, the amount of air, compaction and soil rheology has not been well investigated.

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Table 3.1 An overview of a number of studies reported on factors influencing the mixing process for deep mixing.

<table>
<thead>
<tr>
<th>Referens</th>
<th>Test</th>
<th>Binder</th>
<th>Mixing work</th>
<th>Binder type</th>
<th>Binder content</th>
<th>Rot. speed</th>
<th>Air pressure</th>
<th>Amount of air</th>
<th>Compression/consolidation</th>
<th>Compaction</th>
<th>Soil reology</th>
<th>Mixing tool</th>
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<tr>
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<td>C, W,D</td>
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<td>UC</td>
<td>C,F,G, W</td>
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<td>UC</td>
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<td>L,C,D</td>
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<td>Muro et al. (1987a, 1987b)</td>
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<td>Nishida et al. (1996)</td>
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<td>Tränk &amp; Edstam (1997)</td>
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Test: L=laboratory test, e.g. dough mixer; M=model test; F=field test; UC=unconfined compression test; V=vane test; FC=fall cone test; P=penetrometer tests
Binder: L=lime; C=cement; S=slag; F=fly ash; G=gypsum; D=dry; W=wet
In Japan, part of the binder is often delivered as the mixing tool is being inserted into the soil, particularly when binder in liquid form is used. The binder thus assists the remoulding of soil by the tool. Dry binder may also be delivered in this way in order to utilize the mixing tool more efficiently. To take account of the fact that only part of the binder is delivered during insertion of the mixing tool, the following expression may be used to calculate the number of mixing cycles per column metre $T$ (Hayashi & Nishikawa, 1999):

$$T = \sum M \times \left\{ \left( \frac{N_j}{V_j} \right) \times \left( \frac{W_j}{W} \right) + \left( \frac{N_z}{V_z} \right) \right\}$$

(3.2)

where $W_j$ is the quantity of binder delivered during penetration (kg/m$^3$) and $W$ is the total quantity of binder (kg/m$^3$).

In Sweden, the retrieval rate (mm/rev) of the mixing tool is used as a measure of the mixing time. The blade rotation number or the number of cycles per column metre $T$ can be calculated as

$$T = \sum M \times \frac{1}{s}$$

(3.3)

where $M$ is the number of mixing tool blades, and $s$ is the retrieval rate of mixing tool during withdrawal (mm/rev).

The term blade rotation number has been used over 20 years (used already in the early 1980s by Nakamura et al., 1982) and is stated in the Japanese guideline (CDIT, 2002) and in the European execution standard (prEN 14679).

The effective strain in the mixture is assumed to be a function of the retrieval rate. The intensity of mixing (agitation) is assumed to be a function of the rotation speed. The mixing energy per cubic meter of stabilised soil (in terms of J/m$^3$) is probably a combining key factor. However, the mixing energy is seldom measured and it has not been clearly tested and shown whether the mixing energy is a combining factor.

It will be noted that when investigating the effect of different rotation speeds in Japan, both the effective strain and the mixing intensity are varied. It is therefore difficult to separate these two parameters in Japanese studies, since the rotation speed is increased in order to obtain longer mixing times. The main disadvantage using the term blade rotation number is that the term is closely connected to the present mixing tool geometry. In order to compare different geometries it may be necessary to introduce the mixing energy since different geometries of the mixing tool may perform differently, i.e. it may be difficult to compare paddles with screws.

A conclusion from studies concerning the mixing energy is that the retrieval rate and the number of blades have a significant influence on the strength magnitude and variation. Fig. 3.9 illustrates the changes in strength and coefficient of variation of stabilised soil as the blade rotation number is varied. An approximately logarithmic or a power law relation between the column strength and the mixing work has previously been reported by e.g. Muro et al. (1987a, 1987b), Nishida et al. (1996), Larsson et al. (1999, 2005a,c). However, it is not possible to predict the strength magnitude based only upon the mixing work since the strength in stabilised soil highly depends on the composition and the conditions during the curing period.

![Figure 3.9 Principal changes in strength and coefficient of variation of stabilised soil as the blade rotation number $T$ is varied.](image)

A rational course of action to increase the strength magnitude and to improve the mixture quality is to provide the mixing tool with more blades or construct mixing tools that achieves more mixing work. A higher strength magnitude and a lower variability is of course favourable, but is it economically reasonable to use the mixing work to adjust the strength magnitude compared to use the binder type and content as the primary instrument to adjust the strength magnitude? Furthermore, is it economically reasonable, by means of the mechanical mixing process, to regulate the variation in the properties in the stabilised soil due to different soil compositions? The effect of the mechanical mixing depends on a great number of factors, such as soil composition, binder type and content, time, and the curing conditions. The mixing has different influences in different type of soils. The increase of the strength as a function of the logarithm of the blade rotation number indicates that it may be uneconomical to use the mechanical mixing to adjust the strength magnitude. It is the author's belief that the strength magnitude should be adjusted primary by the binder type and amount of binder.

The influence of the rotation speed is not well investigated. In the Scandinavian countries, the rotation speed is normally between 150 to 200 rpm or as high as possible in order to decrease the time for the installation. The rotation speed has increased over the years from about 60 rpm to 200 rpm. However, there are different opinions about the rotation speed during installation. Principally, a high
Rotation speed is desirable since the intensity of mixing increase and the time for installation decreases. However, the rotation speed is most probably closely related to other influencing factors such as the retrieval rate and the air pressure during incorporation in order to achieve an even binder distribution in relation to the present soil conditions. In Japan the rotation speed is normally lesser, usually 20 to 60 rpm.

Type and quantity of binder

The influence of the type and quantity of binder on the mixing process has only been studied fragmentarily in a number of published studies. There is no question that the quantity of binder affects the strength of stabilized soil, however it can also affect the distribution of the binder and the dispersion of the strength values. Asano et al. (1996) studied the compressive strength and coefficient of variation while varying binder content for three types of binder. The results showed that when using cement slurry, the coefficient of variation fell from ~40 % to ~20 % when the binder quantity was doubled. Nishibayashi et al. (1988) found a similar drop in the coefficient of variation with a doubling of the binder (cement slurry) content as shown in Fig. 3.10a). Nishibayashi et al. (1988) also report results of a study showing that the coefficient of variation increases with the water/cement ratio, suggesting that a greater quantity of mixing water impairs the mixing efficiency (Fig. 3.10b). A reason may be that water is mixed with a soft soil with high water content; the soil breaks up into lumps, which will be entrained by the mixture while undergoing relatively little mixing themselves.

Horpibulsuk et al. (2004) reported results of an extensive series of model and field tests in connection to the ground improvement works for the construction of Saga Airport, Japan. The influence of the binder water/cement ratio, binder content and the mixing work were investigated. As expected when performing field tests, the scatter in the results is significant. However, the large number of tests made an evaluation possible and the tests are highly unique. The water/cement ration of the binder slurry was found to have an effect on the strength and variability. Two groups of mixing states were identified, the workable state and the bleeding state. In the workable state, the mixing functions well at a wide range of mixing work at both high and low binder contents. In the bleeding state, when the water/cement ratio was relatively high, a certain degree of mixing must be performed to create a well-mixed column. Fig. 3.11 shows results where columns are installed by w/c of 80% and 100% of the binder slurry. When the mixing work is low, the binder content has small influence on the column strength. When the mixing work increases the binder content has significant influence. The installation should be performed by a high mixing work using high binder contents. Another observation is that the binder content has a smaller influence when the water/cement ratio increases. An important conclusion from the study is that the strength of the stabilised soil is a combined function of the binder content, water/cement ratio and the mixing work.

Mixing tool design

The influence of the geometry of the mixing tool was investigated during the 1970s and the 1980s, in Japan and Sweden (e.g. Nishibayashi et al., 1984; Nishibayashi et al., 1985; Wikström, 1979). In present reports, the investigations were mainly performed by testing different geometries against each other (e.g. Dong et al., 1996; Abe et al., 1997; Al-Tabbaa & Evans, 1999; Aalto & Perkiö, 2000; Aalto, 2001; Larsson et al., 1999, 2005a,c; Larsson & Nilsson, 2005).

No fundamental scientific studies have been published on the geometry of the mixing tool and its effect on the mixing process in the context of deep mixing. The few studies on tool design that have been reported are somewhat of a “sales” character, often presented by private companies at conferences or in periodicals specializing in equipment. This is of course not very surprising since the mixing tool geometry is one major tool for competition. It is therefore not possible to give general recommendations concerning the mixing tool geometry.

Figure 3.10 Changes in strength and coefficient of variation of stabilised soil as: (a) the binder quantity is varied; (b) the water/cement ratio in the binder is varied (Matsuo et al., 1996 after Nishibayashi et al., 1988)
Figure 3.11 The influence of binder content and mixing work on strength of field stabilised columns: (a) w/c = 80%; (b) w/c = 100% (Horpibulsuk et al., 2004).

In model experiments, it is important to study the impact of scale effects, such as small vertical and horizontal stresses in the mixture. There is a risk of the mixture clinging to the rotating mixing tool during withdrawal. The mixture may also clump together between the blades, particularly when lime is used as a binder. Such effects were not investigated or remarked upon in reported studies.

A common statement is that multiple shaft arrangements generally provide better homogeneity of the stabilised soil than single shaft (e.g. Topolnicki, 2004). It seems reasonable that multiple shafts provide better mixing in the overlapping zone between the columns. However, this statement is based on fragmentary model and field tests (Yoshizawa et al., 1997). With respect to the inherent property variability in stabilised soil, more tests independent tests are required for a general statement.

In the mid 1980s, Inoue & Hibino (1985) and Enami et al. (1986b) presented mixing tools provided with "anti-rotation vanes" which prevent the materials being mixed from rotating along with the tool. These vanes do not rotate in the soil and thus provide resistance to the rotating blades as they pass. The CDM Column 21 mixing tool, as shown in Fig. 3.2c, is provided with mixing blades that rotates in opposite directions. This mixing tool is most similar to mixing tools used in the process industry.

Larsson et al. (2005a) have investigated the influence of the mixing blades in connection with the binder outlet hole. Simple mixing tool geometry was tested as shown in Fig. 3.12. Columns were installed by mixing tools provided with: a) no blades; b) one pair of blades and; c) two pairs of blades and in which the outlet hole was located approximately 200mm above the blades. The columns installed by mixing tool without blades (a) were a circular cavity, 0.1-0.2 m in diameter, which was filled by binder. The mixing tool provided by one pair of blades (b) created full diameter columns, 0.6 m. The mixing tool provided by two pairs of blades (c) could not create full diameter columns even though the blade rotation number is twice as high. The diameter of the columns varied axially from 0.4 to 0.6 m. Consequently, the four mixing blades could not distribute the binder over the whole column cross-section. The conclusion is that the incorporation and spreading of the binder in the cavity formed by the blades in connection with the binder outlet hole are the most important steps of the mixing process (according to section 3.4).

Rheological properties of the soil
The complex rheological properties of soft soils and difficulties of monitoring the mixing process present engineering challenges. The mixing of fine soils with binders is particularly difficult when the soil is extremely cohesive and sticky at moderate to high water contents. The type of soil and its rheological behaviour have a considerable impact on the efficiency of a mixing process. When powdered quicklime, CaO, is used as binder, the rheological properties change very rapidly once the lime comes into contact with the soil. In soft soils, the natural water content is often near the liquid limit. The incorporation of lime causes a rapid dewatering and the mixture becomes more plastic and difficult to work. It is relatively easy, for example, and takes only limited effort to mix cement and dry sand to produce a mixture with small concentration variances. A considerably greater and more intensive mixing effort is necessary to mix lime and cement with cohesive clay. Unfortunately, no extensive investigations have been made into the effect of soil rheology on the mixing process in deep mixing.
Nishida et al. (1996) showed that a higher degree of mixing may be expected in clays with high sensitivity. Larsson et al. (2005a) found that the coefficient of variation, with respect to hand operated penetrometer tests, depends on the quotient between the water content and the liquid limit. As the clay becomes more plastic the coefficient of variation increased. However, there is a need of additional fundamental studies of the influence of soil-binder mixture rheology on the mixing process.

It is difficult to measure and determine the rheological behaviour of soft soils in the partially remoulded state at high shear rates. In Japan, the possibility has been investigated of using the mixing tool as a rheological measuring instrument during the mixing process (Hata et al., 1987; Aoi & Tsuji, 1996; Tateyama et al., 1996). According to the authors, the method may be developed by e.g. measuring the energy input during the mixing process and relating it to soil properties.

Delivery pressure and the amount of air
According to prEN 14679 (2005), “In dry mixing the air pressure shall be kept as low as possible during the mixing process to avoid problems of air entrainment and ground movements”. Aalto (2001) showed in a series of model tests that the amount of air can influence the uniformity and the strength properties considerably. However, the influence of the delivery pressure and the amount of air in the field is not well known. The incorporation and spreading of the binder take part in the cavity formed by the upper pair of blades. The air pressure at the outlet hole is normally not measured or controlled, as it is difficult to measure air borne materials. The air pressure in the binder tank is adjusted with respect to present conditions during installation. The air pressure must be high enough to form the cavity and to create channels up to the ground surface for the evacuation of air. The tank pressure should however not be too high, as this may cause pneumatic fracturing outside the column periphery. A relatively high air pressure, about 200 to 1000 kPa, is common in Sweden, which is notionally high enough to fracture the soil by pneumatic fracturing. The size and the shape of the cavity at the mixing blades depend on the air pressure. At great depth, there is a risk that the cavity will be too small due to high in-situ stresses. The difficulty to create a sufficiently large cavity increases probably with increasing column diameter. A high air pressure and insufficient release of the air pressure could cause heaving and uneven dispersion of the binder and insufficient compaction of the mixture especially close to the ground surface.

Pressure regulation with respect to the depth below ground surface has been tested in Sweden as reported by Bredenberg (1999). However, the binder delivery system is highly complex and therefore the air pressure can not be adjusted rapidly. The Japanese DDM system is designed to evacuate the air via air gathering fins on the shaft of the mixing tool.

Compaction energy
There has been discussion of how soon and how heavily the ground surface may be loaded after the execution of dry deep mixing. It may be important to impose the greatest possible compaction and consolidation stress at an early stage while the chemical reactions are most active. Compaction means densification of an unsaturated soil by a reduction in the volume of voids filled with air, whereas consolidation is densification by the expulsion of water. In Sweden normally no compaction is done apart from that brought about by the mixing tool. Since it is customary in Scandinavia to use air as the transport medium for a powdered binder, large amounts of air are injected into the soil. Unless the air is evacuated, the compacting capacity of the mixing tool may be very important for the mixing process and the strength gain. Even surface compaction may have a significant impact. The interfacial area between the binder and the soil is a measure of the mixture quality and the efficiency of molecular diffusion. The compaction...
energy may thus have an impact on the mixing process, since increased density reduces the distance between the binder particles and the soil particles. In Japan the binder is generally premixed to a slurry, with the result that compaction is of little consequence provided the soil to be stabilised is water-saturated.

Aalto & Perkiö (2000) and Aalto (2001) studied the influence of the consolidation stress during the curing period. Model columns were manufactured by dry deep mixing in the laboratory using cement as binder. With reference to unconfined compression tests, the results showed that the mixing work (retrieval rate) had a significant influence on the strength when the columns were loaded by 40 kPa, Fig. 3.13a. However, the mixing work had minor influence on the strength when the columns were unloaded during the curing period. It must be mentioned that laboratory model columns are made by a low confining pressure that may have an influence on the strength properties. The same problem may appear at shallow depth for the full-scale columns. Similar results were reported by Pousette et al. (1999) showing that the consolidation pressure during curing has a great influence on the strength properties, Fig. 3.13b.

The influence of compaction energy on the strength gain has been a subject for many studies related to shallow subgrade stabilisation (e.g. Bell, 1977). An important factor in compaction is the delay between the incorporation of the binder and the compaction operation ("aging" or "mellowing"). This delay may affect the compaction properties, producing a loss of strength in stabilised soil (e.g. Uppal & Bhasin, 1979; Sweeney et al., 1988; Sivapullaiah et al., 1998; Holt & Freer-Hewish, 1998, 2000). Another used term is the "working period" that indicates the time from incorporation of the binder to compaction where a uniform stabilised material can be achieved. The impact of delay is affected by the composition of the binder, making it difficult to draw general conclusions. However, there are clear indications that the delay between binder incorporation and compaction has varying effects on strength depending on the binder content. The longer the delay, the greater is the strength loss and the lower is the density attainable by compaction. With lower quantities of binder, the delay has negligible effect on density and strength. With high quantities, there is a considerable effect on the strength and compaction parameters.

The time from incorporation of the binder to compaction, or the working time, should be about two hours for cement-stabilised soil according to Sherwood (1993). The working time for lime-stabilised soil is somewhat longer, as long as 72 hours according to British Lime Association (1990). For deep mixing, the working times from a few hours to a few days, is a relatively short time. It is difficult to adjust the loading of the columns to such short periods. The compaction of the binder-soil mixture should therefore be made by the mixing tool and not by loads on the ground surface.

When stabilising peat by e.g. mass stabilisation, 1 m of fill is normally applied immediately after the mechanical mixing process. Åhnberg et al. (2001) studied the delay between the incorporation of the binder and the compaction operation for lime-cement and cement-slag stabilised peat in the laboratory. Fig. 3.13c shows that particularly the cement-slag stabilised peat is affected by the delay between the incorporation of the binder and the compaction operation. It is important to focus on the schedule for loading. If possible, perform the loading in steps where the first loading is performed as soon as possible after execution.

With a rotating tool equipped with paddles or similar arrangements, the effectiveness of compaction depends on the kneading capacity of the tool. It is important that the energy input from the mixing tool is directed axially.

![Figure 3.13 (a) Results of model tests showing the unconfined compressive strength vs. retrieval rate (modified after Aalto, 2001). (b) Evaluated unconfined compressive strength for stabilised peat samples consolidated for different loads (Pousette et al., 1999). (c) Unconfined compressive strength vs. loading delay after mixing (Åhnberg et al., 2001)](image-url)
downwards in the mixture. It is further important that no part of the tool tends to lift the material or cause it to get stuck in the tool and thus be pulled upwards as the tool is withdrawn.

**Author’s opinion**

Table 3.2 shows the outline of the author’s opinions concerning the influence of different factors on the mixing process.

Table 3.2 Outline of a literature review and the authors’ opinions.

<table>
<thead>
<tr>
<th>Factor</th>
<th>Strength variability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mixing tool geometry related to incorporation and spreading of binder</td>
<td>+++</td>
</tr>
<tr>
<td>Rheological properties</td>
<td>+++</td>
</tr>
<tr>
<td>Compaction / consolidation</td>
<td>+++</td>
</tr>
<tr>
<td>Retrieval rate</td>
<td>++</td>
</tr>
<tr>
<td>Number of blades</td>
<td>++</td>
</tr>
<tr>
<td>Binder content</td>
<td>++</td>
</tr>
<tr>
<td>Amount of air</td>
<td>++</td>
</tr>
<tr>
<td>Mixing tool geometry</td>
<td>+</td>
</tr>
<tr>
<td>Type of binder</td>
<td>+</td>
</tr>
<tr>
<td>Rotation speed</td>
<td>-</td>
</tr>
<tr>
<td>Air pressure</td>
<td>-</td>
</tr>
</tbody>
</table>

***+++ Significant and major influence.***

***++ Significant influence.***

***+ Diverged results***

***- No or weak influence.***

### 3.6 Differences and similarities dry - wet

In the author’s opinion, there are more similarities than differences between dry and wet mixing. All deep mixing methods belong to the same physical world. The mechanical systems can be similarly modelled. Stabilised soils are typically $c$'-$\phi$ materials where the strength and deformation properties are strongly dependent on the effective stresses. The original soil has a major influence on the mixing process and the stabilised soil characteristics. Similarly, the mixing processes *in-situ* during the execution belong to the same physical world. Principally, all mixing processes involving the dispersion of fine particles in a liquid can be divided into the four stages according to section 3.4.

However, even though the mixing processes are principally the same there are a number of differences of practical importance. The use of compressed air as the medium for transporting the binder has the advantage that it takes a relatively small amount of binding agent to achieve the requisite strength gain. Given that loose soils already contain a lot of water, it appears logical not to add still more water to the soil, as is done when the wet method is employed. However, the addition of air adds to the difficulty of the mixing process in a material, soft soil, whose rheological properties are already very complex. In the mixing process an air-borne binder complicates the dispersion process with regard to the wetting of lime and cement particles and the breaking up of agglomerates. Wet method is preferred in soils with a high initial strength or low water content. As an example, it is difficult to mix a dry binder in a dry crust.

In general, dry mixing does not require preparations of the work sites such as ample supply of water, plan for the drainage of extruded soil and grout and to clear the site from extruded material (Hioki, 2002; Yasui *et al.*, 2005). Furthermore, there are differences concerning size and weight of equipment and space requirements. The volume of spoil using wet mixing may be high, 50% to 60% of the treated volume (O’Rourke & McGinn, 2004). The dry method creates normally no spoil.

In the Nordic dry method, the dry binder is incorporated into the soil from an outlet hole normally placed in connection to the upper blades on the mixing tool, roughly 0.4 m above the bottom of the mixing tool. Consequently, when a good contact with the underlying frictional layer is required, there may be problems. One solution is to advance and to retrieve the mixing tool several times at the bottom of the column, *i.e.* the mixing tool is moved up and down a number of times. However, the effectiveness of this method is not documented. A simple solution is to incorporate the binder while the mixing tool is penetrating the soil. One reason why this technique is not used in the Nordic countries is that it is difficult and energy-intensive to mix the soil and the binder once they have had time to react together. This problem is particularly acute in the case of lime, as the mixture very rapidly becomes granular.

#### Crater formed holes

Crater formed holes in the ground surface are a frequent phenomenon in dry deep mixing, Fig. 3.14a. They range in depth from a few tens of centimetres to a couple of metres. The reason for this phenomenon is not fully understood but they seem to be frequent and deep if there are a relatively thick and stiff dry crust on low sensitive soils, *e.g.* gyttja. During the incorporation of the binder, the air should be evacuated from the mixture to the ground surface. A low sensitive and sticky soil makes the evacuation difficult and air is accumulated around the mixing tool. The mixing tool and the entrapped air pull the top dry crust with it and when the entrapped air reaches the surface a crater formed hole is formed. There are no documented standard solutions to avoid the formation of crater formed holes but the contractors often have their own solutions. If holes are formed, it is important to fill and compact the holes with a frictional material. If the holes are overfilled without action, there is a risk that the vertical loads creates vaults over the columns and transfer the load to the surrounding unstabilised soft soil, as illustrated in Fig. 3.14b. The consequence may then be larger settlements developed over long time.
It is difficult to mix a dry binder in a dry crust. The lack of water makes the soil stiff and prevents the chemical reactions from occurring. Furthermore, the column installation is normally terminated 0.5 to 1.0 m below the ground surface in order to reduce the risk for blow-outs. The Nordic dry method has been developed in order to improve mixing in the dry crust or in soils of low water contents (Gunther et al., 2004). The method is called “Modified Dry Mixing (MDM)” and the principal is to inject water during the execution in order to generate suitable water content for the mixing process of a dry binder-soil mixture.

**Ground movements**

The installation process may cause relatively large ground movements due to the incorporation of dry or wet binder. High slurry content may result in large ground movements and large amounts of extruded material (e.g. Hirai et al., 1996; Kakihara et al., 1996; Uchiyama, 1996; and Väläaho, 2000). A large number of tests have been performed especially in Japan on this issue. Fig. 3.15 shows records of lateral ground movements during deep mixing works. The figure shows three situations where measuring point lies: (a) on the same level as the stabilisation; (b) over the stabilisation and; (c) under the stabilisation. When the measuring point lies on the same level as the stabilisation, the results indicate that problems may occur when the \( x/L \) ratio is less than 1.5. When the measuring point lies over the stabilisation, the measured lateral movements are very small. However, the lateral movements may be very large when the measuring point lies under the stabilisation as shown in Fig. 3.15c.

It is difficult to determine whether dry mixing causes less ground movements than wet mixing. Kakihara et al. (1996) report a study where dry and wet mixing are compared in parallel and conclude that the lateral movements were larger at wet than dry method.
Few investigations concerning ground movements have been reported in the Nordic countries. Heaves of about 0.2 m have been observed in connection to dry deep mixing, 10 m long columns, as reported by Carlsten and Marxmeier (2000). Relatively large horizontal movements of about 300 mm have been reported in connection to a slope stabilisation work by Viberg et al. (1999). Hallingberg (2005) presents results of ground movements due to the installation of lime-cement columns, installed in a grid pattern, parallel to an existing railroad. The measured vertical and lateral movements in the existing railroad were up to 15 mm and 35 mm respectively ($x/L=0.5$).

The pore pressure in the surrounding soft soil increases due to the installation. Vriend et al. (2000) measured an increased pore pressure of as much as 130 kPa 5.5m from the point of installation. The influence on the surrounding soft soil due to the installation process has been investigated by Shen & Miura (1999) and Shen et al. (2003a,b and 2005). They propose a model where the excess pore pressure is expressed in terms of pressure and pore pressure parameters. The installation process may cause clay fracturing in the range of 2-3 times the column diameter. However, the fractures may improve the pore pressure dissipation and function as channels for the binder to penetrate the soil and thereby improve migration of calcium ions. Fracturing, related to dry method, due to pneumatic fracturing is reported by Larsson et al. (2005a,c). Liu et al. (2005) show that the excess pore water pressure can be reduced by a combination of deep mixing and vertical drains.

An established and an effective method for the reduction of ground movements during installation is the CDM-LODIC method (Kamimura et al., 2005). The principal is to remove soil during installation equivalent to the amount of incorporated binder slurry. The soil is removed by providing the shaft with an earth auger that transports the soil to the ground surface. The displacements can also be reduced by other countermeasures (Uchiyama, 1996): displacement absorbing trenches; air recovery holes; steel sheet piling. Other countermeasures are augered holes filled with bentonite as reported by Ito et al. (1996).

A mixing tool is developed in Japan where the binder is incorporated from the end of the mixing blade (Takeda & Hioki, 2005). The binder outlet hole is placed on the edge of the mixing blade and by which the air-binder mixture is injected towards the mixing shaft. Results presented by Takeda & Hioki (2005) show that the inward-oriented injection reduces the ground movements, makes the air evacuation more effective and eliminates the air leakage to the surroundings.

The sequences of the installation of columns have an influence on the ground movements. Masuda et al. (1996) and Kakihara et al. (1996) have studied different installation sequences and came to the same conclusion. In order to cause the least possible lateral ground movements, the installation should be performed in rows away from the adjacent construction as illustrated in Fig. 3.16. In Sweden, it has been practice to install lime-cement columns according to this sequence in connection to existing railroad embankments. A further step is to install about five rows and then omit five rows etc. The omitted rows are then installed in a later sequence.

![Figure 3.16 Recommended sequence of installation in relation to adjacent construction.](image)

**3.7 On laboratory mixing**

Field studies of the mixing process are difficult and costly to perform. It is therefore appropriate for fundamental studies to be carried out in the laboratory environment using materials with controllable rheological properties and mixing tools with simple geometries. Such work may shed light on which phenomena are likely to be the most relevant for field trials. It should not aim to provide directly applicable data but to yield results that will suggest new approaches to the mechanism behind the real problem. To ensure that new findings are relevant, laboratory tests must keep in close contact with real-world problems. This will also ensure that problems are not over-simplified. Some few scientists state that laboratory-scale auger or paddle mixing should become an integral part of a deep mixing project since they observed similarities between the properties of laboratory-scale test and full-scale test (Al-Tabbaa & Evans, 1999; Larsson et al., 1999; Hernandez-Martinez & Al-Tabbaa, 2004).

However, it is an established opinion that the mixing process cannot be simulated in the laboratory (e.g. Terashi, 1997; Bruce et al., 1998a). The only variables that can be simulated are the type and quantity of binder. The primary purpose of laboratory tests is to tell us whether it is possible to stabilize a soil (e.g. Carlsten, 1991). One purpose of today’s standards is that they should result in a best improvement effect in respect to the degree of mixing which should be considered as a sort of index for the soil in concern (Babasaki et al., 1997). There is an ambition to establish empirical relations, based on accumulated experiences, between the strength achieved in the laboratory and that achieved in the field. However, different laboratories may return different results even when stabilizing the same soil with the same binder (Edstam & Carlsten, 1999). Current laboratory methods for preparing...
test specimens tell us little about the mixing process. Some of the factors affecting the mixing process reported in section 3.5 lie far outside the range of application of deep mixing. Examples are the laboratory preparation of specimens consisting of completely remolded soil and the mixing of specimens many times more than it is reasonable to do in the field. Empirical relationships can only be found if the process parameters of the laboratory tests are within the range of application of the method. Since the factors affecting mixing are probably not mutually independent, there is a serious risk that incorrect conclusions will be drawn if one or more of these parameters are kept constant far outside the application range.

4 PRODUCTION QUALITY CONTROL

Quality assurance and quality control play an important and necessary part of deep mixing works. As for a major part of ground improvement methods, it is necessary to investigate if the improvement will function as intended and to check that the pre-assumed strength and deformation properties have been reached. Thus, the quality assessment must be adapted to the present application and the purpose of deep mixing. For settlement reduction the deformation properties are of main interest whereas for improvement of stability the strength properties are of main interest. For other types of applications, other properties may be of main interest. Quality assessment may also refer to execution control, i.e. the control of the amount of binder incorporated, rotation speed etc. Quality assurance is a process tool that should guarantee that the client receives the ordered product.

Fig. 4.1 shows a flow chart for quality control and quality assurance. The quality control can be divided into laboratory tests, field tests on test columns, quality control during execution, quality verification after execution and follow-up measurements. Laboratory tests are normally performed in advance in order to check if the present soil can be stabilised. These laboratory tests are normally included in the quality assessment process. However, the strength and deformation properties as determined in the field may differ considerably from those for laboratory samples. The discussion concerning laboratory testing and its relation to field properties is extensive and merits an own “State of practice report”. Laboratory testing is not further discussed in this report.

Traditionally, however, the strength with respect to undrained shear strength has been the major checked characteristic. Other commonly checked parameters are the location, length and diameter of columns. The choice of control method depends on what characteristic to be measured, the expected strength in the stabilised soil and the depth to which the control is performed. Different methods may thus be suitable dependent on type of binder. The final design should be based on field measurements and the main difference between quality assessment of dry and wet mixing is the expected magnitude of the strength. An important main task for the quality control is to locate and determine the extent of weak parts. It is important to separate the term quality with respect to the situation it is used, i.e. if it is used to quantify the prediction made of the designer or if it is used to quantify the performance of the mixing device.

Figure 4.1 Flow chart for quality control and quality assurance (modified after CDIT, 2002).
Puppala & Porphaha (2004) and Puppala et al. (2005) have presented the results of an international survey of interest concerning quality control practice. The results showed that in-situ test methods are the most popular and strength was the most important to control. CPT or cone penetration was favoured. SPT has been primarily used in Asia whereas CPT, vane tests and column penetration tests have primarily been used in Europe. The use of geophysical tests is still infrequent. Samples are taken mostly by core sampling and from test pits and excavations. The survey also showed that one of the major limitations of the literature available is the lack of consistency or agreement between studies conducted in Europe, Asia and United States.

4.1 Mixture quality

In a discussion concerning quality assessment, it is first important to define quality as a concept. It is common to use the expression good or bad quality without a clear definition and without a proper sense. Quality control may intend the assessment of strength magnitudes and variability. Quality control may also intend the assessment of the distribution of binders or the variability in strength properties with the aim to control and to judge the mixing process.

The assessment of mixture quality is central to any investigation of mixing. The measurement of degree of mixedness or required mixing time requires the measurement of mixture quality. But when is a mixture well mixed? Traditionally the answer can be that a mixture is well mixed when it is good enough for its purpose. This does not generally mean that it is homogenous. On a sufficiently small scale, practically all mixtures are heterogeneous. Thus, the term homogeneity is difficult to define. Numerous researchers in the process and chemical industries have discussed and written about the concept of homogeneity or the perfect mixture. One example of a definition is offered by Fan et al. (1970), who define a homogeneous mixture as “one in which the content of all constituents is uniform in every part of the mixture”. The term homogeneity may mean different things for different persons and in different industries. Thus, the concept is only useful when associated with a suitable and well defined scale.

When assessing mixture quality it is necessary to distinguish between the distribution of the binders and special characteristics. Many factors are involved besides the efficiency of the mixing process. When assessing the quality of a process it is important to focus on the most important results for the product. To assess the effectiveness of the operation it may be necessary to use a number of indirect measurement methods to assess a specific result, as mixing equipment may perform several functions simultaneously, e.g. breaking up agglomerates and distributing binders.

Generally, an expression for the degree of mixedness is intended as an indication of the variations existing in the mixture. For example, the coefficient of variation can be used as an expression for mixture quality. A sample taken from a mixture can only approximately reflect the distribution of the components of the mixture. The more samples are analysed, the better the approximation will be. When the number of samples analysed is small, the error assumed in the statistical analysis will be large. This error is expressed as a confidence interval defining the bounds within which the statistical value lies. The size of the confidence interval depends on the number of samples analysed and on the statistical reliability. Complete mixing could be defined as the state where all samples extracted from the mixture contain the same proportions of components, or the same properties, as the whole mixture.

Based on results of the determination of the binder content in samples taken from the stabilised soil the mixture quality can be evaluated quantitatively using mixing indices. There are a large number of different mixing indices based on concentration variances defined as (Poux et al., 1991)

\[ M = \frac{\text{The mixing that has occurred}}{\text{The mixing that can be achieved}} \]  

An estimate of homogeneity utilizing concepts of mixing index is based on the ability to estimate concentration variances. On the assumption that sampling errors and analytical errors are independent of each other, the concentration variance of a mixture can be written as (Yip and Hersey, 1977):

\[ \sigma^2 = \sigma^2_{\text{mix}} + \sigma^2_{\text{analysis}} + \sigma^2_{\text{sampling}} + \sigma^2_{\text{purity}} \]  

where \( \sigma^2_{\text{mix}} \) is the variance due to the mixing, \( \sigma^2_{\text{analysis}} \) is the variance due to the analytic method, \( \sigma^2_{\text{sampling}} \) is the variance due to the sampling method and sample size, and \( \sigma^2_{\text{purity}} \) is the variance due to the sample purity. The contribution of sampling method and analytic method to the total error can be substantial and when studying e.g. the effect of a mixing process on homogeneity it is therefore important to be able to estimate the influence of the analytic method and the sampling method and to ensure that samples are not contaminated.

The efficiency of mixing can also be assessed by comparing the strength of laboratory prepared samples with the strength of samples taken from the site. This type of definition of the quality assessment is based on the assumption that the strength and deformation properties obtained from tests on laboratory prepared specimens are the best achievable. The design must be based on experiences of differences between lab and field.

Porbaha et al. (1999) propose the use of a reliability-based quality index based on the resistance ratio. The resistance ration \( R \) can be defined as the dimensionless ratio of the measured strength to the design strength.

\[ R = \frac{\text{Actual strength}}{\text{Design strength}} \]
The quality index describes the number of standard deviations separating the best estimate of \( R \) from its limiting value of 1.0 according to the relation

\[
\lambda = \frac{\bar{E}[R] - 1}{\sigma[R]}
\]  
(4.4)

where \( \bar{E}[R] \) is the expected value and \( \sigma[R] \) is the standard deviation. The resistance ration \( R \), as a quality index, considers thus the pre-assumed strength in the design.

It is the author’s opinion that the efficiency of mixing should be based on the variations existing in the mixture, e.g. the coefficient of variation. It may be confusing to use the strength magnitude as a measure since a high strength doesn’t necessarily mean that the variations existing in the mixture are low. The strength magnitude depends mainly on the binder type and content and less on the efficiency of mixing.

### 4.2 Quality control

The installation process is supervised by continuous monitoring and recording of a number of parameters. According to CENT C 288 the execution control shall include:

a) penetration and retrieval speed of mixing tool;

b) rotation speed of the rotating unit(s) of mixing tool;

c) air pressure (in case of dry mixing);

d) feed rate of binder/slurry.

The torque or some other energy-related parameter is normally measured, however not in the Scandinavian countries. The installation process control may also involve the recording of mixing depth, start time, time at bottom, finish time, grout mix details, grout injection pressure, total grout injected, the density of the slurry. Pore water pressures, vertical and lateral movements are sometimes measured during installation.

### 4.3 Penetration methods

There are numerous of publications on penetration methods used for the quality control of stabilised soil. A wide range of penetration methods has been used or developed in Asia and Europe to assess the quality of stabilised soil. These methods are conventional or specially designed penetration methods such as pushing, pulling, rotary or dynamically driven methods. Based on an extensive survey, Puppala et al. (2004a) recently proposed protocols for execution and evaluation of the test results for SPT, CPT and pressuremeter tests. Further surveys of interpretations of test results and case studies are presented in Puppala et al. (2004b). The Swedish column penetration test is recently reviewed by Axelsson & Larsson (2003).

**Column penetration tests**

In the early stage of the lime-cement column method development, it was concluded that static penetration tests were unsuitable for production quality assessment because of the small test-volume and the difficulty of knowing the exact position of the probe (Boman et al., 1980). In cooperation with the Swedish National Road Administration, Torstensson (1980a, 1980b) developed the lime column probe, which is the most used column penetrometer test today (Fig. 4.2a). A further development of the column penetration test was presented by Chalmers University of Technology during the early 1990s. The test was called reversed column penetration test (Ekström, 1992). The shape of the probe was retained but the direction of the testing was changed. The method was further developed in which the probe is installed during the mixing process, termed preinstalled reversed column penetration test, Fig. 4.2b (Holmqvist, 1992). The objective with reversed column penetration tests is that the probe is kept in the centre of the column during the test. The probe is normally placed during column manufacturing under the mixing tool and the wire runs through the kelly bar. When the column is manufactured, the probe remains below the column and the wire runs through the whole column up to the ground surface.

Column penetration tests are normally performed according to the Swedish guidelines (SGF, 2000), also described in prEN 14679 (2005). In this test, the probe should be as wide as possible, preferably 100 mm smaller than the column diameter. The test is executed by pressing the probe down into the centre of the column at a speed of 20 mm/s with continuous recording of the penetration resistance. A centre hole is prebored when necessary in order to facilitate verticality. According to Ekström (1994), columns up to 12-15 m length with compressive strength up to 600-700 kPa can be tested with this method. Local parts of high strengths may be penetrated by dynamic impact. The probe may be provided with several blades in order to improve the guidance of the probe and to test a larger part of the column cross section (Halkola, 1999).

**Figure 4.2 (a) The lime column probe (Torstensson 1980a, 1980b). (b) Probe for reversed column penetration test (after Holmqvist 1992).**
The undrained shear strength of the stabilised soil is evaluated using a bearing capacity relation according to Equation 3.6. Boman (1979) proposed a bearing factor that is approximately equal to 10 for a probe with the area of 100 cm². Holm et al. (1981) and Broms (1984) proposed, however, that the bearing factor should be 11, based on a comparison with pressuremeter tests. According to the Swedish guidelines, the bearing factor $N = 10$ can be used (SGF, 2000). However, Wiggers & Perzon (2005) state that the bearing factor $N = 20$ should be used in stabilised peat.

There exists no standard with respect to the dimensions on the probe. Therefore, the shape and area may differ and it is therefore important to consider the area of the probe used in every project (Axelsson, 2001). A joint problem with most of the sampling and testing techniques is that only the centre parts are tested. If the strength properties vary over the column cross-section the results obtained from column penetration tests may be misleading.

Recent experiences show that column penetration tests are considered reliable as the primary quality test with reference to the uniformity and continuity of the columns as discussed by Axelsson & Larsson (2003). Several investigations indicate that the reversed column penetration is not a reliable test since the wire disturbs the mixing process. The problem is, however, easily solved letting the column machine install the probe after the manufacturing. The reversed column penetration test, installed in this way, is considerably more reliable than the conventional column penetration test since the verticality is ensured and a larger force can be applied during testing.

**Dynamic penetration tests including SPT**

Different types of dynamic penetration test may be used where a probe penetrates the stabilised soil using a hammer weight. The number of hammer drops to reach a desired depth provides a rough index of the strength of the stabilised soil. In Finland, the weight of the hammer is 8 kg and the fall height is 575 mm (Huttunen et al., 1996). The diameter of the conical point is only 20 mm. In Japan, lighter equipment is used where the weight of the hammer is 5 kg and the fall height is 500 mm (Hosoya, 1997).

Standard penetration test SPT may be the most widely used field test method for soil investigation and is also used in stabilised soil especially in Asia. However, the method can not be regarded as well established in Europe. The SPT probe is driven into the soil by dropping a weight from a constant height. The number of blows to penetrate the soil 300mm may be taken as a rough measure of the strength of the stabilised soil. The results must be regarded as coarse and should only be used as a relative measure. The method should only be used as a secondary test method.

**Cone penetration tests CPT**

During the 1990s, cone penetration tests (CPT) were occasionally used in Sweden for determination of the
strength parameters and the uniformity of lime-cement columns. CPT provides continuous measurements of the penetration resistance, sleeve friction and pore water pressure at the tip of the probe. However, CPT tests have not been established in Sweden because of difficulties in maintaining verticality and due to the limited volume tested. The problem is similar to that of the static penetration tests, which was rejected as quality control in the 1970s. In Finland and Norway, however, CPT tests are commonly used (Halkola, 1999; Want et al., 1999) and the method is used as a primary test method is several countries in Europe, e.g. England. CPT is also often used as a control method in block or masstabilised peat, e.g. Máčik et al. (1999). As for other types of stabilised soils there are problems reported concerning the evaluation of the test results and the strength properties and that the penetrometer has a tendency to deviate from the column at relatively shallow depths (e.g. Huidén, 1999). The undrained shear strength is evaluated, as for the column penetration test, by a bearing capacity relation according to Equation 4.6. The choice of a proper bearing capacity factor has been discussed and the lack of reference methods makes the choice difficult.

A common problem with CPT is difficulties keeping verticality. High bar friction makes the problem worse. Porbaha et al. (2001b) presented a promising development of CPT where the bar friction is decreased by circulating mud water along the bar during penetration. The method is named FRICON (FRictionless CONe). Porbaha et al. (2001b) also presented results of a comparison of in-situ strength evaluated by FRICON and results from shear and compression tests on core samples. The results indicate that the correlations between direct shear tests and unconfined compression tests \((q_u/2)\) with cone are 22-23 and 18 respectively. The bearing capacity factor \(N_q\) is thus 22-23 related to direct shear tests and 18 related to unconfined compression tests.

The seismic cone penetration test has been tested by Hansson et al. (2001). The seismic cone evaluates both strength and stiffness properties where the shear wave velocity is measured by two sets of geophones mounted in the cone. The method may be an interesting alternative to geophysical down hole tests.

CPT is not commonly used in Japan due to the problems connected to high strength stabilised soil. It is difficult to penetrate cement columns and even more difficult to keep verticality. CPT is applicable for low strength stabilised soil (Hosoya et al., 1997).

In Finland, a combined static-dynamic penetration test has been used for high strength columns where the capacity for CPT has been exceeded (Halkola, 1999). The rod is rotating and pressed down and when the machine’s capacity is exceeded a ram start to hit the rod.

The Swedish weight sounding method, also a static penetration test, is used occasionally as secondary complement to the column penetration test. As for the standard penetration test and CPT the weight sounding method is considered as unsuitable due to the small volume tested. However, the small probe may be suitable in inclined soundings to test the strength over the column cross-section and to test the overlapping between columns installed in panels (Larsson, 2005).

Vane tests

The vane test is an important and useful method of determining the shear strength of soft soils. In the early 1980s, a special vane test for lime columns, composed of thick vanes, was developed in Finland (Halkola, 1983). The diameter and the height of this vane are 132 mm and 65 mm, respectively. The blades are thicker than normal, 6 mm next to the shaft and 3 mm for the outer half of the wings. A modified vane has also been used in Norway as reported by Braaten et al. (1999) and Aaboe et al. (2000). According to Halkola (1999) the maximum undrained shear strength, which can be measured, is 200 kPa. According to Axelsson & Larsson (1994) the correlation between the vane test and the column penetration test is good when the column shear strength is roughly 100 kPa. However, this vane has only rarely been used in Sweden, as the vane may disturb the stabilised soil during penetration. The method can therefore underestimate the strength considerably as reported by Boman (1979) and Axelsson & Larsson (2003).

Soil/Rock Sounding, Total Sounding and Rotary Penetration Test

There exist a number of different rotary penetration methods where the test principles are to measure a number of parameters during penetration and drilling. There are different types of drilling bits used.

Soil/Rock probing is a Swedish method where the thrust, drilling rate rotation speed, torque, spoil water flow and water pressure may be measured. The method is used for preboring before column penetration tests or in high strength columns when the strength is too high for other methods. According to Ekström (1994), the undrained shear strength can be roughly estimated from the penetration resistance using a bearing capacity factor of 20. In high strength columns, the factor should be higher.

In the rotary penetration test, which is developed in Japan, the tip resistance is measured together with the torque and water pressure by a specially designed probe. The strength can be estimated empirically from the measured drilling speed, rotation speed, thrust and torque (Hosoya et al., 1997; Porbaha, 2002; Porbaha & Puppala, 2003). However, there are empirical constants that must be determined related to each of the four measured variables.

Jelisic & Nilsson (2005) have tested the Total sounding (a development of the Soil/Rock probing) in cement columns at Husby, Sweden. Based on a somewhat circumstantial reasoning the undrained shear strength is evaluated from the sounding resistance. The reasoning by Jelisic & Nilsson
(2005) is mainly based on the validity of a bearing capacity relation, similar as for CPT tests. However, the failure mechanisms may be principally different since the total sounding cuts the soil by a drill bit. The sounding resistance is not measured at the tip which makes the influence of the bar friction uncertain. It is remarkable that it was possible to test 18 m long columns since it is common that probes are deviating out of the columns.

4.4 Load tests

Dead load- and plate load tests
The compression modulus can be determination by full-scale embankment load tests. Test results have recently been presented by e.g. Craft (2004), Stewart et al. (2004), Allén et al. (2005), Cali et al. (2005a) and Eriksson et al. (2005). These types of tests are not common but should be performed more often. Load tests can also be performed on single short columns (Kivelö, 1998). A vertical load is applied in steps on a column similar to the load test of piles. Raju and Abdullah (2005) show results of 4-column plate load tests performed in Malaysia.

Plate load tests have been tested in order to estimate the deformation properties of lime-cement columns in Sweden (Baker et al., 1997, Baker, 2000; Baker et al., 2005). Two plates are used that are connected by a steel wire, Fig. 4.4. The lower plate and the steel wire are installed during the column installation, similar procedure as for the reversed column penetration test. The wire is loaded by a hydraulic jack.

Pressuremeter test
Pressuremeter test is used to evaluate the strength and compression modulus in columns. A pressuremeter is installed in a drilled hole in the centre of the column. The pressure and displacement of a membrane is measured in radial directions. The strength properties are evaluated using cylindrical cavity expansion models.

The advantages are that the compression modulus can be evaluated for a relatively large volume and high strength columns can be tested. However, the results are often difficult to analyze as reported by Hughes et al. (2001). The test is relatively time consuming and thereby relatively expensive. Few records of pressuremeter tests are available. Hughes et al. (2001) performed pressuremeter tests and unconfined compression tests on core samples taken in the same bore hole, Fig. 4.5. The results illustrate a typical example of problems of comparing different methods when a number of test conditions differ. Cali et al. (2005) presents results where pressuremeter tests and compression tests on field samples have been performed in parallel. No correlation between the compression tests and in-situ pressuremeter tests was evident. However, in the US the pressuremeter test is considered as an excellent tool for quality control (e.g. Hughes et al., 2001; Esrig et al., 2003).

Hydraulic conductivity tests
The hydraulic conductivity of lime-cement columns is seldom measured even though the property has an influence on the rate of deformation. Very few records are available from field tests. Pressure-permeameter tests, which are similar to pressuremeter tests (Baker, 2000), have been carried out by Pramborg & Albertsson (1992) on stabilised soil. Baker et al. (1997) and Baker et al. (2005) have performed tests similar to those used for flow tests in boreholes in a rock mass.
4.5 Geophysical methods

Geophysical methods have been used, mainly on experimental level, in order to obtain an initial overall assessment of the quality of the stabilised soil which can assist in optimising the control programme. Geophysical methods involve seismic methods (in hole, crosshole and downhole logging) and electrical resistivity. The dynamic compression and shear modulus are evaluated by measuring the compression and shear wave velocities, respectively. The average velocity of seismic waves between a source and a receiver is measured. The tested volume is large in comparison with penetration tests. Small defects may thus have little influence on the evaluated results. Major advantages are that geophysical test are non-destructive and can be performed in the same material at several occasions. The interpretation of geophysical measurements is a somewhat complex process and a great deal of practical experience is required of the operator. The basic of geophysical methods is not treated in this report since there exists numerous of excellent references. A concise description of seismic testing is presented by Massarsch (2005).

There is an obvious optimism concerning geophysical methods since quality control normally is complex and relatively expensive. In the authors’ opinion, there is a somewhat uncritical attitude concerning the possibilities to evaluate strength and deformation properties in stabilised soils from geophysical measurements. A relatively large amount of geophysical measurements on stabilised soil have been performed in the lab. However, it is important to evaluate their relationship with in-situ tests. One obstacle to the use of geophysical techniques is the unfamiliarity of geotechnical engineers regarding what they can expect from geophysical tests and the relation to static geotechnical strength and deformation properties as emphasised by Foti & Butcher (2004).

Seismic methods

Fig. 4.6 shows results of previously presented studies concerning the relation between the unconfined compressive strength vs.: a) S-wave velocities and; b) P-wave velocities. The results concerning the relation between the unconfined compressive strength vs. S-wave velocities (Fig. 4.6a) indicate that there is a strong correlation between the S-wave velocity and the unconfined compressive strength in the low strength interval $q_v < \approx 2\text{MPa}$. In the higher strength interval, $q_v > \approx 2\text{MPa}$, the results concerning the relation between the unconfined

Figure 4.6 Unconfined compressive strength vs.: a) S-wave velocities and; b) P-wave velocities.

![Figure 4.6 Unconfined compressive strength vs.: a) S-wave velocities and; b) P-wave velocities.](image)

Figure 4.7 Unconfined compressive strength vs. S-wave velocities and P-wave velocities of specimens bored from three sites (Nishikawa et al., 1996).

![Figure 4.7 Unconfined compressive strength vs. S-wave velocities and P-wave velocities.](image)
compressive strength vs. P-wave velocities (Fig. 4.6b) indicate a relatively weak correlation. Fig. 4.7 shows results of the relation between the unconfined compressive strength vs. S-wave velocities and P-wave velocities of specimens bored from three sites in Japan (Nishikawa, et al., 1996). P-wave and S-wave velocities increase with strength of improved soil but the correlations are very weak. Yesiller et al. (2000) explain the poor correlation between P-wave velocity and unconfined compressive strength by the fact that strength is highly affected by the composition of the material, including voids, imperfections and defects and is not as closely correlated to wave velocity. The influencing factors are probably not independent, which may make the obtained results site specific. S-wave tomography by means of cross-hole measurements has also been performed in cement stabilised soil by Hane & Saito (1996).

The Integrity test, similar as for tests of concrete pile, is used in Japan on columns with unconfined compressive strength over 1MPa (Tamura et al., 1996; Futaki & Tamura, 2002).

The problem of comparing the relation between S-wave velocities and other test methods in the field are shown in Fig. 4.8. The figure shows the relation between the unconfined compressive strength of core drilled specimens vs. S-wave velocities measured in the field. As expected the scatter related to compression tests on small core drilled samples is large.

Unfortunately, there are few records presented of the correlation between static deformation properties vs. S-wave and P-wave velocities. Yesiller et al. (2000) obtained relatively good relations were found between the P-wave velocity and the compression modulus and densities respectively as shown in Fig. 4.9. Wave propagation occurs along the fastest path in the soil and is therefore strongly correlated to the stiffness. Hird & Chan (2005) developed a method of establishing a correlation between unconfined compressive strength and shear wave velocity or maximum shear modulus of stabilised soil.

Massarsch (2005) states that it is possible to determine static deformation modulus at small strains from seismic tests based on the observations that the rate of loading during seismic tests is comparable to that of a conventional static test. The main reason for the differences between the seismic and the static modulus is the strain magnitude, which must be considered in an analysis.

**Electrical resistivity**

Electrical resistivity measures the soil resistance to electrical current flow. The technique may be applicable as a quality control method for stabilised soils. Tests in cement stabilised soil and discussions concerning the applicability have been published by Imamura et al. (1996), Tamura et al. (2002), Staab et al. (2004) and Staab et al. (2005).

Imamura et al. (1996) obtained correlations between the electrical resistivity vs. the unconfined compressive strength and the water content of core samples from jet-grouted columns. According to Imamura et al. (1996), it is possible to interpret the quality of the stabilised soil, if the relation between the electrical resistivity vs. strength and the water content etc. is known. Staab et al. (2004) show that depend on soil type, cement content, water content, and time after mixing. An important conclusion as stated by Staab et al. (2005) is that there appears to be strong correlations between electrical resistivity and strength when the cement content is constant. When the cement content varies the correlation is weak. Staab et al. (2005) emphasize that further field tests are required to evaluate how well this technique works on actual stabilised soil.
4.6 Sampling

Coring

Laboratory strength tests on core samples are normally the primary test methods for the quality assessment of cement treated soil. Coring can be conducted by different types of methods (double or triple tube), depending on the strength magnitude. The main reason for core sampling is difficulties performing penetration tests in high-strength stabilised soil.

Coring is sensitive to the sampling device and technique (Porbaha, 2002) and should be supplemented by other test methods. The small size tested may cause difficulties in the evaluation of the test results due to the large variability in the small scale and size effects. Several investigations have reported size effects where the measured strength decreases with increasing sample size (e.g., Futaki et al., 1996; Hosoya et al., 1997). The strength in cement-stabilised soil may be significantly affected by weak layers, cracks and micro-cracks. Triaxial tests are preferable to unconfined compression tests. In Japan, it is recommended that the samples should be at least 76mm in diameter and 150mm in length.

The continuity of the stabilised soil is evaluated by a visual observation and the RQD value (Hosoya et al., 1997). The quality is also evaluated by the average value and the coefficient of variation from compression tests. The problems related to the evaluation of test results from compression tests on core samples are illustrated by Fig. 4.10. If all results are considered the mean may be low and the variance high due to disturbances. If the better cores are selected the mean will probably be too high. The correct distribution is uncertain and must be assumed.

At the Boston's CA/T project more than 7000m length of core was drilled (Lambrechts & Nagel, 2003). The extensive and unique experiences from this project resulted in a series of valuable tips, e.g., triple-tube barrel should be used. Also Sugawara et al. (1996) report experiences and present a developed core sampler. Burke & Sehn (2005) review results of compression tests on core samples and discuss experiences from two projects in USA.

Wet grab samples

Wet grab sampling is a sampling technique where samples are taken directly after the execution of wet deep mixing methods. A sampling device is lowered down to the sampling depth where still wet, liquefied, soil-binder mixture is captured and brought to the ground surface. The young wet mix is poured into cylinders for laboratory tests. Wet grab sampling has been common especially in Europe and USA.

The main uncertainty may be how representative the recovered samples of the stabilised soil are as discussed by Bruce et al. (2000). There are investigations indicating that the strength obtained from wet grab samples is roughly half the strength obtained from core samples (Taki & Yang, 1991; Burke, 1998). The opposite condition has been reported by others (Lambrechts & Nagel, 2003; O'Rourke & McGinn, 2004). Large property variability has been reported, often a coefficient of variation of 50 % (Burke et al., 2001; Vriend et al., 2000). If there is large scatter in the results, it is difficult to determine the influence of sampling, curing etc. The shape of the wet grab sampler seems to have an influence since clay clumps may be blocked as discussed by O'Rourke & McGinn (2004). Burke & Sehn (2005) review results of compression tests on core samples from a number of projects in USA.

Auger boring and Piston sampling

Auger boring has been used to take disturbed samples in lime-cement columns for the determination of the binder content. However, this method can only be used when the column strength is relatively low and is therefore not commonly used. Piston sampling has also been used (Braaten et al., 1999; Lawson et al., 2005) but it is difficult to get undisturbed samples even when the column strength is low.
**Test pits**
Sampling, testing and visual examination can be carried out in columns, which have been excavated in open test pits. The maximum depth without special means is roughly 2-4m depending on the site conditions. Test pits are popular since they provide simple observations of column shape, diameter, overlap etc. The rate of unstabilised or weak parts over the column cross-section may be evaluated by pocket penetrometer tests or similar (e.g. Futaki & Tamura, 2002). A major disadvantage is that the binder dispersion over the cross section and the strength and deformation properties may vary considerably over the column length, which is common in layered soils (Larsson, 2001). The tests performed on a shallow depth may therefore only provide limited information.

**Extraction of columns**
Equipment for the extraction of complete columns was developed in the 1970s (Broms et al., 1978; Boman, 1979). This method, as shown in Fig. 4.11, has been used in several projects in the 1990s (Rogbeck, 1997; Axelsson & Rehnman, 1999; Holm et al., 1999; Hansson & Eriksson, 2000; Rogbeck et al., 2000; Kujala & Lahtinen, 1988; Want, 1999). The extraction of whole columns facilitates sampling over the whole column length and over the whole column cross-section. The column strength may be controlled by simple test methods like pocket penetrometer.

Figure 4.11 Extraction of an 8-metre long lime-cement column using a split tube sampler (Axelsson, 2001)

In some cases, undisturbed samples have been taken for compression tests. However, there are major problems obtaining undisturbed samples. The same problem has been observed in block-stabilised peat where Mácsik et al. (1999) report a case where most of the samples were fractured. The quality assessment is often performed only by visual examination.

A simple equipment, developed from the reversed column penetration test, where a load plate is installed at the bottom of the column, may be used to extract whole columns when the strength is high (Åbjörn & Linnér, 1995).

**4.7 Visual examinations**
Visual examinations cannot be used for quality assessment since the visual impression is difficult to quantify and is not necessarily equivalent to the binder distribution (Larsson, 2001). Visual judgements are associated with human senses, and are therefore highly individual and subjective. For example, it is difficult for the human sight and feeling to detect strength variations for high strength material. However, since visual examination is simple it is tempting to judge the column quality based on individual visual assessments. The visual examination may be a complementary tool to other types of testing.

**4.8 Control of the verticality and diameter**
There exist no simple and established method for the control of the verticality and diameter of columns (e.g. Axelsson, 2001). The diameter is normally controlled in open test pits or by the extraction of whole columns. The verticality can be controlled by measuring the centre of the columns at some stages in a deep excavation.

In the case of overlapping columns the verticality has a determining influence on the function. According to the Swedish guideline (SGF, 2000), the inclination tolerance should be in the interval $0.6^\circ - 1.1^\circ$ (1:100–1:50). The overlap between two columns is normally 50-100mm. As a result, even when the columns are installed within the given tolerances, the overlap may cease to exist with lengths exceeding 2.5-5m. The development of methods for the control of the verticality is a subject for further studies as emphasized by e.g. Axelsson (2001) and Massarsch & Topolnicki (2005).

**4.9 Construction performance**
Follow up measurements are normally performed in order to verify the behaviour of the ground improvement. During construction, e.g. embankments or excavations, the lateral displacements and settlements are measured by surface markers, settlement plates, settlement forks, tell-tales, flexible hoses and inclinometers. The stress-strain behaviour between the columns and the surrounding soft soil can be measured by e.g. earth pressure cells, strain gauges and vertical borehole extensometers. The pore pressures should be measured but it is rather unusual in
Sweden. It is important to measure the parameters included in the used design model.

4.10 On the observational method
The execution of deep mixing, including the design process, is normally performed in several iterative phases. In Sweden, this process is often named active design, however often without a clear definition. As an important step of the active design process, test columns are regularly installed in advance or at the start of a project (e.g., Holm, 2000). Primary design, often based only on laboratory tests and established experiences, should be followed up by an observational approach. The observational method (Peck, 1969) entails the following items as expressed by Ladd (1986):

- “That the designer select appropriate quantities to be monitored during construction; make predictions of their magnitudes based on the working hypothesis adopted for design (i.e., the most probable conditions) and also for the most unfavourable likely conditions; and develop suitable actions or modifications to handle all significant deviations from the design hypothesis.

- That the field instrumentation provides reliable, timely measurements of the quantities to be observed.

- That the engineer evaluates these data to ascertain the actual condition and have the ability to implement corrective actions during construction.”

The observational method as stated by Ladd (1986) is focused on the mechanical design and not as much on the design of the execution process. prEN 14679 (2005) provides a somewhat extended approach including the design of the execution process. “The following instructions shall be given prior execution:

a) reporting procedure for unforeseen circumstances, or conditions revealed that appear to be different from those assumed in the design;

b) reporting procedure, if an observational method of design is adopted;

c) notice of any restrictions such as construction phasing required in the design;

d) a schedule of any testing and acceptance procedures for materials incorporated in the works.”

An accurate application of these instructions will result in an increased confidence for deep mixing. However, it is easy to discuss active design, more difficult to put into practice. The observational approach must include magnitudes of the decisive parameters and a plan for steps to be taken if values are likely to be exceeded. It is important to emphasise that the observational method must not be a substitute for design. Too often poor design is disguised by routine measurements that are not followed-up.

The observational method gives the opportunity to regard the present design from an actual behaviour. The verification of the design shall be based on actual observations and measurements during construction. The observational method is suitable for deep mixing since it normally is difficult to predict the properties in the improved soil and it is possible to control the actual behaviour. According to Eurocode 7, EN 1997-1:2004, the observational method may be appropriate to apply when prediction of geotechnical behaviour is difficult. The observational method involves a probabilistic approach where the probability of the behaviour of the construction shall be within established limits. The observational method can thereby not be a substitute for design and a “design as you go” method.

The design of a control and measuring system must be based on the possibility to perform the measurements sufficiently accurate with respect to the instrument, personal and interpretation. The design must also be regarded as a contractual and organizational issue. The measurements and observations shall result in decreased uncertainties and thereby be the basis of necessary changes in the design. In this matter there are two types of uncertainties related to measurements and observations; statistical uncertainty is normally due to inadequate number of data and; professional uncertainty due to insufficient knowledge (paradigm) of the actual behaviour of the stabilised soil. These two types of uncertainties are briefly discussed in the following two sections, 4.11 and 4.12 respectively.

4.11 Discussion on the extent of testing and sampling
There exists no universal well-established approach concerning the extent of testing and sampling. The extent should be determined depending on: the type and purpose of the ground improvement work; test and sampling method; and the nature of the stabilised soil variability.

The Swedish guideline SGF (2000) gives general recommendations concerning the extent of columns that should be tested, based on the size of the improvement works. However, this recommendation is based on the conditions that the design undrained shear strength is maximum 100 kPa and that the total factor of safety is higher than 1.0 for the corresponding unstabilised construction. Normally, about five penetration tests are performed in parallel in order to obtain an average value. There has been a discussion concerning a statistical treatment of the test results, e.g., Halkola, (1999). However, there has not been any straight-forward statistical methodology adopted in the Scandinavian countries.

The stabilised soil should not be tested too early since the chemical process has a large influence on the strength and deformation properties during the first weeks after
installation. It is however desirable to test columns in an early stage of the work in order to facilitate improvement of the design (Axelsson & Rehnman, 1999). Furthermore, it may be difficult to test the stabilised soil after several weeks if the strength is high.

The Japanese guideline CDIT (2002) formulates a procedure for the evaluation of results from compression tests on core samples where the coefficient of variation is considered. The following relation shall be satisfied:

\[ \bar{u}_{ck} \leq \bar{u}_f (1 - KV) \]  

(4.5)

where \( \bar{u}_{ck} \) is the design standard strength (average unconfined compressive strength), \( \bar{u}_f \) is the average unconfined compressive strength of the core samples, \( V \) is the coefficient of variation and \( K \) is a random variable that determines the confidence. Futaki & Tamura (2002) state the following values of \( K \) as a function of the number of samples \( N \) with 90% confidence.

<table>
<thead>
<tr>
<th>N</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4-6</th>
<th>7-8</th>
<th>9-12</th>
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<tbody>
<tr>
<td>K</td>
<td>1.8</td>
<td>1.7</td>
<td>1.6</td>
<td>1.5</td>
<td>1.4</td>
<td>1.3</td>
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Fig. 4.12 shows a graphical presentation of the relation (the reduction in strength \((1 - KV)\) as a function of the coefficient of variation for varying number of samples tested). As the number of samples increases, a higher strength can be utilised. The figure shows that the reduction in strength \((1 - KV)\) is high when the coefficient of variation is high, cf. Figs. 4.12 and 5.4. The figure also reflects the discussion concerning stabilised soil variability in chapter 3. It is important to consider the variability related to the present mechanical system. If the mechanical system can be considered as an averaging process the utilisation of variance reduction has a major influence on the strength reduction.

The European standard, “Eurocode 7: Geotechnical design” states “If statistical methods are used, the characteristic value should be derived such that the calculated probability of a worse value governing the occurrence of the limit state under consideration is not greater than 5%”. Consequently, a statistical treatment of test data on stabilised soil should be used in the European countries as well.

4.12 Discussion on evaluated strength- and deformation properties

Stabilised soils are typically \( c' - \phi \) materials where the effective stresses have a major influence on the shear strength. However, it is difficult to determine the cohesion and the angle of friction and little effort has been done on this area. Furthermore, it is difficult to measure the pore pressure that is necessary for an effective stress analyses. Traditionally, stabilised soil has been considered as a soil where the strength parameters are approximately independent of the stress path to failure. This approximation can be related to early work on a design model for lime columns. In consequence, sounding methods are normally calibrated with respect to unconfined compressive strength obtained from compression test on core samples. There is a definite need for extensive data from triaxial tests.

There is a tradition to evaluate the undrained shear strength from column penetrometer tests and CPT tests in the Scandinavian countries. The undrained shear strength in the improved soil \( c_{u,col} \) is normally assessed using a bearing capacity relation

\[ c_{u,col} = \frac{(q_c - \sigma_{vo})}{N_k} \]  

(4.6)

where \( q_c \) is the probe resistance, \( \sigma_{vo} \) is the effective overburden pressure, and \( N_k \) is the bearing capacity factor. The magnitude of the bearing capacity factor \( N_k \) depends on the soil, the size and shape of the probe, the soil plasticity, depth, the overconsolidation ratio, and the performing of the correlation etc. Thus, the bearing capacity depends on the failure mechanism during the sounding and the way of performing the correlation to theoretical assumptions or experiments. Very few studies have been published concerning accurate choice of bearing capacity factors related to different load situations and applications. There is, therefore, a significant risk for bias in the evaluation of strength and deformation properties.

As for a number of penetrometer methods such as column penetrometer tests, CPT and weight sounding tests, an essential disadvantage is the uncertainty of the evaluated properties of the improved soil. When assessing the undrained shear strength, the undrained stress condition is assumed, strengthened only by a relatively fast sounding procedure. The correct definition of undrained condition based on constant volume during shearing is normally not
shown or investigated. Since the end of the 1980s, cement-based binders are normally used, and the improved soil behaves more like an unsaturated frictional material. Improved soil behaves like an overconsolidated clay or clay bearing silt, i.e., the induced pore pressure is relatively low during undrained triaxial tests (Baker, 2000; Ekström, 1994; Åhnberg, 2005). Baker (2000) explains this behaviour by the fact that the improved soil is not fully saturated. For stiff soils a degree of saturation around 97% results in a very low generated pore pressure and the improved soil depends thereby on the confining pressure. In a present published paper, Åhnberg (2004) shows that the backpressure during undrained triaxial tests has a significant influence on the evaluated shear strength. Thus, the undrained failure condition can be questioned and the effective stress approach may be more appropriate. Furthermore, with cement as binder, the improved soil becomes brittle and the failure pattern around the probe is uncertain. The main failure mechanism may be cracking rather than shearing as assumed. It may therefore be more appropriate to use the compressive strength instead of the shear strength as the evaluated strength property in the improved soil.

In the Scandinavian countries, dry deep mixing is often used to reduce settlements in very soft soils. The deformation properties are in these cases of main interest. However, the traditional fixation of testing and evaluating the strength with respect to penetration tests have resulted in a limited knowledge concerning deformation properties of full scale stabilised soils and the interaction with the over-lying structure. Furthermore, there is a lack of simple reliable test methods. Sounding methods can be used to evaluate the uniformity and to roughly estimate strength properties. The estimation of deformation properties is even more insecure and not recommended. Empirical relationships exist between strength and stiffness but their correlation is generally poor (e.g. Åhnberg et al., 1995; CDIT, 2002; Massarsch & Eriksson, 2002; Massarsch, 2005). The determination of the strength and deformation properties requires in-situ tests, geophysical tests or load tests. Few test methods measure directly deformation properties and there are not enough experiences from in-situ tests such as pressuremeter tests and geophysical tests. It is common to use experience-based values on modulus of elasticity and permeability in design. Increased knowledge concerning deformation properties and the development of test methods would definitely increase the use of deep mixing for the foundation of e.g. houses.

It is important to consider the objective of the ground improvement when choosing test methods (Axelsson & Larsson, 2003). For settlement reduction, the deformation characteristics are of main interest; for improvement of stability, the strength characteristics are crucial. Production control methods for the assessment of the cohesion, internal friction, and the pore pressure in the stabilised soil are required. It should be investigated whether existing test methods can be useful for the assessment of cohesion and internal friction. Fundamental studies concerning improved soil behaviour in the context of theoretical soil mechanics, and studies concerning spatial variability are required in order to provide help to designers. At present, direct determination of the strength and deformation properties may require special equipments and evaluations, such as pressuremeter tests or the use of load tests.

5 VARIABILITY IN DEEP MIXING

Execution and mixing affects physical and chemical properties, including uniformity, strength- and deformation properties, and permeability. As for natural soils, stabilised soils have relatively high inherent property variability. There are several reasons why it is important to consider property variability in stabilised soil. The variability is of importance in relation to the probabilistic design and in connection with quality assessment since the variability has an influence on the requisite test and sample sizes. Furthermore, knowledge of the factors affecting property variability is important when developing the mixing process. There exist no general guidelines concerning sufficient variability or mixing quality related to different applications. According to prEN 14679 (2005) “The rotation speed of the rotating unit(s) and the rate of penetration and retrieval of the mixing tool shall be adjusted to produce sufficiently homogeneous treated soil”. This statement provides a real challenge in design. The variability related to strength properties should be small when the columns are used to increase the stability of high embankments, slopes and excavations. Larger variability is accepted when the main purpose for the ground improvement is to reduce settlements. However, in order to meet the statement the discussion must be more modulated.

The link between the mixing process and the mechanical system is visualised in Fig. 5.1. The mixing process generates a mixture with certain properties. The stabilised soil has relatively high property variability and the average and variability, e.g. the coefficient of variation, can be evaluated by e.g. test samples. However, the measures of the average value and variability are related to the test method, the test- and sample size as well as the statistical sample size. The analysis of the problem includes an analysis of the mechanical system and the related scale, the “scale of scrutiny”. The variability of a property in the stabilised soil is related to the scale of scrutiny and the correlation structure, i.e., how the property varies from one point to another in space. The variability related to the mechanical system may thus differ from the variability obtained from samples and testing. As for many geotechnical applications, the test and sample sizes may be small compared to the scale related to the mechanical system. The design values can subsequently be evaluated by taking the concept of failure probability and safety into consideration.
The variability in stabilised soil is unfortunately not well investigated. However, this section briefly discusses the concept “scale of scrutiny” and “spatial variability”, and discusses the link to probabilistic design. Published records concerning strength variability are reviewed.

5.1 On the mechanical system
Stabilised soil is complex with respect to its spatial strength- and deformation properties. The stabilised soil as natural soil is in-homogenous; the properties are anisotropic and non-elastic. It can generally be said that the higher the strength of the stabilised soil, the more brittle the material becomes. The mechanical system includes present failure mechanisms where the system behaviour depends on; the present type of system (e.g. series or parallel; the type of element (e.g. elastic-plastic, elastic-brittle) and; the type of loading (even distributed or determined by the deformations). The knowledge of the stabilised soil mechanical system is limited, owing to the fact that the strength- and deformation properties are dependent on a largely number of factors. The uncertainty is both due to insufficient understanding of the individual failure processes in the singular, groups or rows of columns and the extent of the interactions between failure processes and between the stabilised soil and the surrounding soil.

According to Swedish practice, stability calculations are based on the assumption that the columns and surrounding soft soil behave as a composite material (SGF, 2000). Failure is assumed to occur along a slip surface through the columns and the surrounding soil. An averaging failure model, a parallel system is assumed in the full structure scale as well as the smaller column section scale (Fig. 5.2). The averaging model can be assumed, provided that the characteristic undrained shear strength of the stabilised soil is less than approximately 150 kPa. The stress-strain relationship is assumed to be elastic-plastic, i.e. all parts of the column cross-section interact at failure.

For high strength stabilised soil and/or stabilised soil containing a high amount of cement, other failure models may be appropriate to use. Honjo (1982) applied the bundle model on stabilised soil. A bundle model is somewhat between the average model and the weakest link model (Fig. 5.2). Omine et al. (1998) proposed a failure model combining the weakest link model and the bundle model. The model called “a combined model” considers the amount of cracks in the stabilised soil and expresses the size effect on the strength, caused by the extent of potential cracks.

In order to establish the mechanical properties of the governing parameters, the variation of the average values have a great influence on the profitability of failure. The mechanical system depends on the variability of the governing parameters. Further knowledge may be added to the design in order to guarantee that the variance is not over estimated when the behaviour may be described as a parallel system and a sum of elastic-plastic elements. The possibility to utilise variance reduction is important to investigate for improved soil (further discussed in section 5.4). Since the mixing process has a significant influence on the average value and the variance it is motivated to discuss variability in deep mixing.
5.2 The scale of scrutiny

The quality of stabilised soil depends on the scale on which it is examined. The stabilised soil may appear to be homogeneous when viewed by eye. When observed under the microscope, however, it becomes apparent that the particles are not evenly distributed. Thus, the degree of homogeneity can only be determined once a suitable scale of scrutiny has been established. The appropriate scale of scrutiny itself depends on the application and the adherent mechanical system.

In deep mixing where molecular diffusion (the migration of calcium ions) is involved as a mixing mechanism, it must also be remembered that mixing may continue even though mechanical mixing has stopped. In such a process mixedness improves with time (however, not necessarily the strength variability), and hence the time at which the mixture quality is assessed will influence the result. In practice the appropriate scale of scrutiny has to be estimated, however the concept is a useful one for defining mixture quality. Furthermore, an established scale of scrutiny limits the size or volume of samples to be analysed in order to assess the quality of a mixture.

According to Dankwerts (1953), the scale of scrutiny can be defined as “the minimum size of the regions of segregation in the mixture that would cause it to be regarded as imperfectly mixed for a specified purpose”. Consequently, according to Dankwerts definition the scale of scrutiny is related to the mixture quality, e.g. the variability in a certain property. The scale of scrutiny is thereby determined by the particular application and the adherent mechanical system. The variability of a property in the improved soil of importance for the design is related to the scale of scrutiny and the correlation structure of the parameter.

There are a number of parameters influencing the strength properties in stabilised soil; the characteristics of the binder; the characteristics of the soil; the mixing process; and the curing conditions (e.g. Terashi, 1997; Babasaki et al., 1997). Similarly, there are variabilities associated to each of these parameters. Furthermore, the variability associated to each of the parameter is connected to a certain scale, i.e. the parameters may influence in different scales. The scale may differ considerably and what seems to be a scatter in a certain scale may be a trend in a smaller scale. Consequently, it may be important to determine the correlation structure for each parameter related to the scale of scrutiny.
Fig. 5.3 shows an example where the stabilised soil variability is separated into three categories, three different scales. The first category, “the whole structure”, the strength distribution in the stabilised soil is governed by the soil composition, lithological heterogeneity and inherent spatial variability, uneven binder distribution, and different conditions during curing. The second category, “column group”, is manifested in the form of varying binder contents due to uneven binder distribution during the mixing process. This variable binder distribution is associated with the difficulty to achieve an even binder distribution. Soil properties and curing conditions are assumed not to vary. The variable binder distribution results in variabilities in the axial direction in the column but also variabilities between adjacent columns. The third category, “column segment”, is ascribed to inherent spatial variability, which is the variation in properties from one point to another due to the performance of the mixing process. The soil properties and the curing conditions are assumed to be constant. The binder distribution in the axial direction is considered to be constant. The mixing process is considered to have an influence on the strength variability in a relatively small scale, associated to the binder dispersion over the column cross-section. In practice, it is difficult to separate the three types of categories. When performing testing or sampling it is important to consider the scale of the testing and the scale associated with the mechanical system. However, the size of probes and samples are normally much smaller than the scale of scrutiny. In consequence, the evaluated variability may be related to a small scale, e.g. the column segment.

**The full structure**
The strength distribution in the stabilised soil is governed by the soil composition and properties, lithological heterogeneity and inherent spatial variability, uneven binder distribution between columns, and different conditions during curing.

**Column group**
(in a horizontal plane)
The strength distribution in the stabilised soil is governed by uneven binder distribution. Soil properties and curing conditions do not vary.

**Column segment**
The strength distribution over the column section is governed by the mixing process. The soil properties and the curing conditions do not vary. The binder distribution in axial direction is constant.

Figure 5.3 Strength variability in different scales: (a) full structure; (b) column group; (c) column segment (Larsson et al., 2005b)
5.3 Variability in deep mixing

Fig. 5.4 shows results of a number of reported studies in which the coefficient of variation of compression test results was evaluated. All data are from tests on samples taken from in-situ stabilised soil. These results show that the evaluated coefficient of variation varies considerably. According to Suzuki (1982), Honjo (1982), Kawasaki et al. (1984) and Larsson et al. (2005a, 2005b) the dispersion in plan over column cross sections is of the same order. The coefficient of variation is larger for on-land work compared to marine work (Terashi, 2005). It is difficult to compare the results between different studies since the test conditions may differ considerably. Large values may be an indication of factors such as sampling techniques and soil variability, e.g. depth trends, rather than large inherent stabilised soil variability. However, based on the reported results it is possible to question a common belief that the mixing quality is better using wet mixing. It is the author’s opinion that this belief often is based on hearsay and visual observations and not actual measurements. Wet and dry method should be tested in parallel under equal conditions before any statements can be done.

It is unusual that the number of samples of stabilised soil extracted in a Scandinavian project is sufficient to permit evaluation of the coefficient of variation in e.g. compression tests. However, Axelsson & Rehnman (1999) and Braaten et al. (1999) have presented results of a large number of compression tests on specimens from lime-cement columns. The variability in strength is often high when the columns are installed in layered soils, which is common in Sweden. With column penetration tests, which test a large part of the cross-sectional area of a column in-situ, large variations may also be obtained between different columns. Kujala et al. (1985) report standard deviations in the order of 15 – 60 % of the mean.

Jelisic (1999) investigated mass-stabilised peat with a number of different test methods and reported a coefficient of variation as high as 90%. This extreme variation in strength is probably not related only to the strength variability, but also to the different methods and evaluation methods used, unfortunately not investigated by Jelisic (1999).

5.4 Spatial variability

Traditionally, deep mixing structures have been designed deterministically, i.e. the structure has been divided into a number of zones of uniform material. Each zone is characterised by a single set of material properties, one single analysis is performed and by a single factor of safety. The procedures for selecting nominal values of stabilised soil properties are not well-defined. There exists a great difference between different engineers, some use the mean value and some use a conservative value or even the lowest measured value. The different and highly diverged procedures may lead to uncertain designs. The factor of safety is incorporated without a reasonable consideration of the present mechanical system.

Since stabilised soil exhibits relatively large variability, a more realistic alternative is to consider the natural variability of the stabilised soil properties. The only methodology available today that can connect physical and probabilistic requirements is reliability-based design (e.g. Kulhawy & Phoon, 2002). The uncertainties in mechanical properties can be quantified and the reliability can be enhanced.

Figure 5.4 Coefficient of variation evaluated from compression tests in a number of reported studies.

   Honjo (1982)
2. Mori et al. (1984)
4. Saitoh et al. (1996)
5. Hosomi et al. (1996)
7. Futaki et al. (1996)
8. Asano et al. (1996)
10. Mizutani et al. (1996)
13. Braaten et al. (1999)
expressed in terms of probabilities of failure. Reliability-based design requires knowledge of the present mechanical system, an established probable magnitude of each parameter and an estimate of the variability and the variation in space. However, reliability-based design is not well established for deep mixing and one reason is that it is not obvious how to predict and evaluate strength and deformation properties in stabilised soil in relation to the mechanical system. The probability distribution is also important in relation to the production quality assessment with reference to the strength- and deformation properties, since it has an influence on the extent of the control. The spatial correlation structures have an influence on the distance between the tests in order to obtain representative measures of properties. The spatial correlation structures have thus an influence on the requisite test- and sample sizes.

The need of reliability-based design in deep mixing was emphasized as far back as the early 1980s by Honjo (1982), but has still not been set into practice. According to Kitazume (2002), reliability-based design will be introduced in the coming revised technical standard in Japan planned in 2006. The discussion has also been taken up by e.g. Porbaha & DeMillio (2004) who emphasised the importance to incorporate reliability concepts in design of deep mixing projects. The knowledge concerning property variability is limited and there is a need for additional test at different conditions.

Reliability-based design (or probabilistic design) requires knowledge of the soil variability related to the scale of scrutiny in the present mechanical system. The variability of a soil property can be described statistically by the probability distribution, where the point-wise variability is represented by the mean value $\mu$ and the standard deviation $\sigma$. These parameters vary in space. The description of the spatial variability requires an additional parameter, the scale of fluctuation $\theta$, which estimates the distance within which the soil property shows relatively strong correlation (Fig. 5.5). The property variability by means of spatial correlation structures is an assessment of the variation of properties from one point to another in space. The spatial correlation structures describe the dependence between the properties as a function of the distance between the points. The scale of fluctuation $\theta$ is evaluated thorough the determination of semivariograms and the author recommends Deutsch (2002) for the modelling of spatial correlation structures.

If the mechanical system can be considered as an averaging process, (i.e. the average value of the investigated parameter is of importance for the mechanical system) the utilization of variance reduction has a major influence on the probability of failure. The average value always has a smaller variability than the single point variability. However, the average value must not be mixed up with the mean value in series of tests at single points.

When there is dependence between the samples, as described by the spatial correlation structure, there is a corresponding reduction of the variance of an average over a length, area or volume. The variance of the mean value $\sigma^2_{\text{ave}}$ over volume $V$ can be correlated to the sample

![Figure 5.5 Illustration of variability in stabilised soil: (a) strength as a function of depth; (b) normalised strength as a function of depth where the trend is removed; (c) a column cross-section illustrating the zone around a tested point or volume where the tested property shows relatively strong correlation.](image)
variance $\sigma^2$ using the variance reduction factor $\Gamma(L)$ through (Vanmarcke, 1977):

$$\sigma_{\text{mean}}^2 = \Gamma(L) \times \sigma^2$$  \hspace{1cm} (5.1)

Vanmarcke (1977) proposed a simplified expression for a “standard” reduction factor $\Gamma(L)$ given by

$$\Gamma(L) = \begin{cases} \frac{1}{\theta} & L \leq \theta \\ \frac{L}{\sqrt{\theta}} & L \geq \theta \end{cases}$$  \hspace{1cm} (5.2)

The variable $L$ is a characteristic length representing the size of the average length, area or volume, i.e. the scale of scrutiny. The above expression is valid for the one-dimensional case. By assuming separable correlation structures in the two- and three-dimensional case, the variance reduction factor can be expressed as the product of its one-dimensional components.

Fig. 5.5a shows the strength variation with depth from a sounding in stabilised soil. There is a large-scale trend due to different soils in the profile and strength increase with depth. The point-wise variability becomes large if the large-scale trend is not removed in an analysis of the soil variability. Besides measurement errors, this may be a possible reason for the large variances reported in stabilised soils according to Fig. 5.4. Fig. 5.5b shows the same sounding result and the scale of fluctuation $\theta_f$ in the vertical direction when the large-scale trend has been removed.

Results of about ten analyses at two sites presented by Honjo (1982) showed that the vertical correlation distance is about 0.4 to 4.0 m in cement stabilised soil. Pobaha et al. (1999) analysed the vertical correlation distances at two sites by six analyses of the results from CPT tests. The correlation distances are 0.23m and 0.62m for the two sites. Futaki et al. (1996) show the results of two analyses and concluded that the vertical correlation distances were 0.8 and 0.1 m respectively. None of the studies considered large-scale trends in the analyses.

Nilsson (2005) shows results of a large number of evaluated correlation structures in lime-cement columns. The vertical correlation structures by means of autocorrelation functions were performed with respect to column penetration tests. The scale of fluctuation is in the range 0.12 to 0.63 m when the scale trends (due to soil stratifications and strengths increase with depth) were removed in the analyses. Similar results have been reported by Hedman & Kuokkanen, (2004).

Navin & Filz (2005) have studied the horizontal correlation structures by means of autocorrelation functions between different columns at the I-95/Route 1 project in the USA. Data of compression tests on core samples from 206 columns were included in the analyses, which make the study unique. Analyses were performed on columns installed by wet and dry methods, respectively. The analyses from wet method indicate that the correlation distance is about 12m. However, the analyses from dry method did not reveal a spatial correlation, i.e. the horizontal correlation distance is less than the distance between two columns.

A large number of analyses of lime-cement column cross-sections indicate that the horizontal correlation structure is geometrically anisotropic around an axis of symmetry caused by the rotating mixing device (Larsson et al., 2005b). The scale of fluctuation differs in the radial and orthogonal directions as illustrated in Fig. 5.5c. The scale of fluctuation in the radial direction $\theta_r$ is in the range 0-110 mm and the scale of fluctuation in the orthogonal direction $\theta_o$ is in the range 0-300 mm. A consequence of the relatively small values of the scale of fluctuation is that small samples only represent the stabilised soil in a narrow volume around the sample and not a major part of the column cross-section. In order to assess an average value from e.g. small 50 mm diameter samples, a number of samples must be taken from different parts of the column cross-section.

The uncertainty in the evaluation of the average value is probably decisive for the total uncertainty since the part, which is governed by the spatial correlation structure, is significantly decreased by the variance reduction (Olsson, 1986). Small zones of low strengths may thus have little influence on the average strength. The variability obtained from small samples may thus overestimate the variability related to the mechanical system. As pointed out by Druss & Young (2005), the occurrence of measured data, which fell below the minimum strength requirement, may give the client an unrealistic negative assessment of quality of the improved soil. If the mechanical system can be considered as an averaging process, tests should be performed in a relatively large scale. If the scale of scrutiny corresponds to a column cross-section, the test size should not be much smaller. Sampling for the determination of the binder content must be done in a sufficiently large scale, preferable the same size as the scale of scrutiny. Sampling by auger boring or core drilling may be unsuitable due to the small scale and the uncertainty of knowing the position of the sampler device.

Further investigations should be performed on the correlation structures between spatial averages of the stabilised soil properties in column groups, i.e. assessing the correlation structure for column groups. It is of interest to study the volumetric correlation between the average strength- and deformation properties within different columns, separated by the column spacing. The correlation structures between spatial averages affect the testing frequency. The spatial averages should be determined by test methods where a relatively large part of the column cross-section is tested. Based on the probe size, the column penetrometer test is the most suitable test method today for this purpose. A method for the assessment of the correlation structure between spatial averages is presented by Vanmarcke (1977). The correlation structures for stabilised soil properties in the full scale will facilitate the determination of a proper selection of partial factors.
6 FUTURE NEEDS IN RESEARCH AND DEVELOPMENT

This report is intended to form a basis for the discussion on future needs in research and development concerning execution and quality control, according to Session 6 at the conference. However, the research needs concerning execution and quality control are not independent of the research needs concerning stabilised soil properties and design. It is the author’s opinion that increased knowledge concerning the stabilised soil mechanical system related to stabilised soil properties is by far the most important research area (including increased knowledge concerning stabilised soil properties). However, a number of future needs in research and development related to execution and quality control are listed below.

- Increased knowledge concerning stabilised soil variability, i.e. consideration of the nature, measurement and statistical characterisation of stabilised soil variability. Furthermore, investigate how probabilistic and stochastic methods of analyses may be used to assess the effects of stabilised soil variability and, via increased understanding, the influence on design and construction.

- A modulated discussion concerning the concept "Sufficient mixing quality", including a clear definition of the concept quality.

- Development of test methods testing a large part of the column cross-sections. Most of the methods used today test a too small volume related to the most common mechanical systems.

- Development of simple test methods for the evaluation of deformation properties.

There are other areas of future needs in research and development connected to execution and quality control. However, it is the author’s opinion that most of these areas are of minor importance in a comparison with the above listed areas.

7 CONCLUDING REMARKS

The Grouting and Deep Mixing conference IS-Tokys’96 (Yonekura et al., 1996) provided the first extensive worldwide compilation of deep mixing techniques. The development of deep mixing methods exploded in Japan during the 1980s and 1990s. The Japanese have developed a deep mixing toolbox for a wide range of applications. In the Scandinavian countries, the volume of deep mixing projects increased rapidly from the end of the 1980s in connection to large infrastructure projects. During the 1990s, the use of deep mixing methods increased in Europe and in United States. In spite of extensive experiences and a large number of published conference papers there were a lack of papers presented in scientific Journals concerning deep mixing. The period from 1999 at the time for the SD-conference in Stockholm to 2005 may be characterised as a period where deep mixing methods have consolidated and matured. Deep mixing has been accepted world vide.

There are a large number of test methods used for the quality assessment of stabilised soil. The reasons are the great differences in strength and deformation properties. According to Porbaha (2002), “The most commonly cited barrier to the use of deep mixing (DM) technology is practitioners’ lack of confidence in their ability to assess the quality of the finished DM product”. Unfortunately, this condition is still prevailing. There exists a large amount of papers on quality control methods and case studies. However, there are some disagreements on conclusions of tests reported and very few studies are published in scientific journals. Rathmayer (1997) stated in a regional report at IS-Tokyo’96 that “the only reliable test method today is total sampling, managed by lifting up of the entire column”. Unfortunately, this statement is still prevailing. There is still a lack of simple reliable methods.

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9 REFERENCES


