Deep Mixing Method in Japan

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ABSTRACT: The Deep Mixing Method (DMM), a deep in-situ soil stabilization technique using cement and/or lime as a stabilizing agent, was developed in Japan and in the Nordic countries independently in 1970s. Due to its wide applicability and high improvement effect, the method has gained increased popularity in many countries. The method has been successfully employed in thousands of projects and the volume of improved soil from 1977 to 2010 exceeded 100 million cubic meters in the Japanese market alone. In the past three to four decades, traditional mechanical mixing has been improved to meet changing needs. Also new types of technologies have been introduced in the last 20 years and put into practice; e.g. high pressure injection and hybrid of mechanical and high pressure injection. The design procedures for various infrastructures were standardized by responsible organizations in Japan and revised several times. The manuscript presents a State of the Art on the Deep Mixing methods in Japan that covers the machinery, design, construction and quality control and assurance of the Deep Mixing Method.

Keywords: Ground Improvement, Deep Mixing Method, Design, Execution, Quality control and quality assurance

1. INTRODUCTION

It is difficult to locate a new infrastructure on a good ground due to the over-population in urban areas throughout the world. Renovation or retrofit of old infrastructures should often be carried out in the close proximity of the existing structures. Good quality material for constructions is becoming a precious resource to be left for the next generation. Due to these reasons and environmental restrictions, ground improvement is becoming a necessary part of infrastructure development projects both in developed and developing countries. This situation has especially been pronounced in Japan, where many construction projects must locate on soft alluvial clay grounds, artificial lands reclaimed with soft dredged clays, highly organic soils and so on. These ground conditions would pose serious problems of large ground settlement and deformation and/or instability of structures. Apart from clayey or highly organic soils, loose sand deposits under water table would cause a serious problem of liquefaction under seismic condition. Such foundation ground is called a 'soft ground' and needs to be improved.

The Deep Mixing Method (DMM), a deep in-situ soil stabilization technique using cement and/or lime as a stabilizing agent, was developed in Japan and in the Nordic countries independently in 1970s. Due to its wide applicability and high improvement effect, the method has gained increased popularity in Europe, Asia and in the USA. The method has been successfully employed in thousands of projects and the volume of improved soil from 1977 to 2010 exceeded 100 million cubic meters in the Japanese market alone. In the past three to four decades, traditional mechanical mixing has been improved to meet changing needs. Also new types of technologies have been introduced in the last 20 years and put into practice; e.g. high pressure injection technique and hybrid of mechanical and high pressure injection technique. At present, the deep mixing method includes the mechanical mixing technique, high pressure injection technique and hybrid of mechanical and high pressure injection technique. The design procedures for various infrastructures were standardized by responsible organizations in Japan and revised several times.

As the State of the Art on the Deep Mixing Method is published recently that covers the mechanical properties of stabilized soils, design, execution and quality control and assurance of the above three techniques (Kitazume and Terashi, 2013), here the machinery, design, construction and quality control and assurance of the mechanical mixing technique in Japan are briefly introduced.

2. CLASSIFICATION OF DEEP MIXING METHOD

In the deep mixing method, soft soil is stabilized in situ with binder without compaction. The deep mixing method (DMM) has usually been applied to improvement of soft clays and organic soils for various purposes such as stability, settlement reduction, excavation support and seepage control (Coastal Development Institute of Technology, 2002). The deep mixing method originally developed in Japan and in the Nordic countries has now gained popularity in the worldwide market. During the past three to four decades, a variety of deep mixing processes have been proposed by contractors as their proprietary techniques. The mixing processes are classified in Table 1, which follows the classification system first adopted by Bruce *et al*. (2000) but is expanded to include the additional systems available in 2012 Kitazume and Terashi, 2013). The column from the left shows the method of introducing the binder either by Wet (binder-water slurry) or Dry (dry powder). The second column shows the driving mechanism of mixing tools. The third column shows the type of mixing tool and its location. For the high pressure injection, the second and third columns are combined. The fourth column shows the name of techniques followed by the country or region which was originally developed. The fifth column shows the roots of techniques either originally developed for deep mixing or modified from diaphragm wall or trench cutter.

The techniques in which dry binder is blown pneumatically into a ground are called the dry method of deep mixing. The dry method employs mechanical mixing which consists of vertical rotary shaft(s) with mixing blades at the end of each shaft. In the penetration and/or withdrawal stage, binder is injected into the ground. The mixing blades rotate in the horizontal plane and mix the soil and the binder. In one operation, a column of stabilized soil is constructed in the ground. The two major techniques for the dry method are Japanese DJM and Nordic dry method. The standard DJM machine is a dual shaft machine and both the penetration/withdrawal speed and rotation speed are fairly slower than the Nordic single shaft machine. The DJM has been used extensively in Japan and the Nordic dry method is used mostly in Nordic countries but also used in the other parts of the world in lesser extent. It seems that both Japanese and Nordic dry methods have not experienced substantial change during the last two to three decades.

The techniques in which binder-water slurry is pumped into a ground are generically called the wet method of deep mixing. The wet method, as shown in the table, has a variety and new techniques are continuously appearing on the market.

Table 1 Classification of deep mixing based on mixing process)

Binder Type	Type of shaft	Position of mixing	Representative system	Origin
Dry	Vertical rotary shaft	Blades at bottom end of shaft	DJM (Japan), Nordic dry method (Sweden)	Deep mixing
Wet A	Vertical rotary shaft	Blades at bottom end of shaft	CDM (Standard, MEGA, Land 4, LODIC, Column21, Lemni2/3) (Japan), SCC (Japan), Double mixing (Japan), SSM (USA), Keller (Central Europe), MECTOOL (USA)	
Wet B	Vertical rotary shaft assisted by Jet	Blades and high pressure injection at bottom of shaft	JACSMAN (Japan), SWING (Japan), WHJ (Japan), GeoJet (USA), HydraMech (USA), TURBOJET (Italy)	
Wet C	High pressure injection at bottom of shaft		Jet grouting – single fluid, double fluid, triple fluid (Japan), X-jet (Japan)	
Wet D	Vertical rotary shaft	Auger along shaft	SMW (Japan), Bauer Triple Auger (Germany), COLMIX (France), DSM (USA), MULTIMIX (Italy)	Diaphragm wall or Trench cutter
	Horizontal rotary shaft	Vertical mixing by Cutter mixer	CSM (Germany, France)	
	Chainsaw, Trencher	Continuous vertical mixing	Power Blender (Japan, shallow to mid-depth, down to 10 m), FMI (Germany, shallow to mid-depth), TRD (Japan, down to 35 m)	

* DJM: Dry Jet Mixing method, CDM: Cement Deep Mixing method, LODIC: Low Displacement Control method, JACSMAN: Jet And Churning System MANegement, WHJ: Waterfront Hybrid Jet mixing method, SMW: Soil Mixing Wall method, CSM: Cutter Soil Mixing, TRD: Trench Cutting Re-mixing Deep wall method.

The techniques in Wet A in Table 1 were originally developed for deep mixing and share the same fundamental mechanism with the dry method mentioned above. The equipment has a single to eight vertical rotary shafts equipped with cutting edge, blades or paddles at lower part of each shaft. Further modifications/improvements of the basic techniques are purpose oriented. The CDM-LODIC added a continuous auger at the upper portion of shafts to remove a certain portion of original soft soil during penetration and withdrawal phases in order to reduce the displacement of nearby existing structures. The CDM-MEGA, CDM-Land 4 and CDM-Lemni 2/3 are aimed to improve productivity either by expanding the diameter of mixing blades or by increasing the number of shafts. The CDM-Column 21 and CDM-Double-mixing are employing sophisticated mixing tools to improve the uniformity of soil-binder mixture (Cement Deep Mixing Method Association, 1999).

The techniques in Wet B in Table 1 are hybrid of mechanical mixing and high pressure injection mixing. In these techniques central portion of deep-mixed column is produced by the same process as those of Wet A and the diameter of which is governed by the size of horizontally rotating blades. In addition to the mechanical mixing, the equipment in this group has the nozzle(s) at the outer end of rotating blade(s) from which the high pressure cement slurry is injected outward to create ring-shaped treated soil and expand the overall diameter of deep-mixed column. All the methods except the JACSMAN employ horizontal jet and hence the outer diameter of ring-shaped soil depends on soil condition and the applied pressure. The JACSMAN employs a pair of nozzles at two different levels: upper nozzle inclines downward and lower one inclines upward in order to make two jets collide at prescribed point to maintain the constant outer diameter of ring-shaped stabilized soil. The hybrid method is effective when the overlapping of adjacent deep-mixed soil columns is important or when the contact of stabilized soil to the existing structure is required.

The techniques grouped in Wet C in Table 1 are high pressure injection mixing methods called jet grouting. The high-pressure binder slurry with/without the aid of other high pressure fluids is injected into a soil at high velocities from the nozzles located at the bottom of drill shaft. They break up the soil structure completely and replace/mix the soil particles in situ to create a homogeneous mass. When the fluids or binders are injected horizontally, the diameter of completed stabilized soil is difficult to control and that depends on the injection energy and the original soil conditions. The X-jet technique injects the binder from the two nozzles at different levels and two jets are designed to collide each other at prescribed radius in order to create a stabilized column with uniform diameter. As the size of the equipment is extremely smaller than the mechanical deep mixing equipment, the technique is quite useful in a situation with space and head room restrictions.

The techniques grouped in Wet D in Table 1 seem to stem from the techniques for diaphragm wall construction or for trench cuttings and are recently modified to meet the deep mixing requirements. Mixing is carried out by various processes such as continuous or discontinuous augers along shaft, cutter blades rotating around the horizontal shaft, or continuous transportation and mixing of soil-binder mixture by chain-saw type mixing tools.

Figure 1 shows the deep mixing machines. Figures 1(a) and 1(b) show the Japanese dry method, DJM and the Japanese wet method, CDM mounted on a special barge for marine construction (Wet A). Figures 1(c) and 1(d) show the high pressure injection (Wet C) and the hybrid of mechanical and high pressure injection (Wet B), respectively.

3. DEVELOPMENT OF DMM

Research and development of the deep mixing method in Japan was initiated by the Port and Harbour Research Institute (PHRI) of the Japanese Ministry of Transport. The concept of lime stabilization of marine clays was first publicized in a technical publication of the PHRI in 1968. When the feasibility of the method was confirmed in the early 1970s, the research and development of the deep mixing method was accelerated. The subjects of the R/D included 1) investigation of the lime and cement reactivity of marine clays, 2) development of equipment which was capable of providing constant supply of binder and reasonably uniform mixing at depth, 3) understanding engineering charac teristics of stabilized soil, and 4) establishing design procedure.

By the extensive laboratory tests on a variety of clays, it was found that most of Japanese marine clays easily gained strength of the order of 100 kN/m^2 to 1 MN/m^2 in terms of unconfined compressive strength (Okumura and Terashi, 1975; Terashi *et al*., 1977). Terashi and Tanaka at the PHRI continued the study on the engineering properties of lime and cement stabilized soils (Terashi *et al*., 1979, 1980, 1983) and proposed a laboratory mixing test procedure. The procedure was welcomed by Japanese researchers and engineers, and essentially the same procedure was adopted by the Japanese Society of Soil Mechanics and Foundation Engineering in 1981 as its Draft Standard JSF: T31-81T. The draft was later officially standardized by the Japanese Society of Soil

(a) Dry mixing method for on-land works

(b) Wet mixing method for marine works

(c) High pressure injection method

(d) Hybrid of mechanical and high pressure injection method

Figure 1 Mixing machines of the deep mixing method

Mechanics and Foundation Engineering in 1990 and experienced minor revisions in 2000 and 2009. The researches were followed by the extensive studies by the research group of Takenaka Co. Ltd. (Kawasaki *et al*., 1981; Saitoh *et al*., 1985; Niina *et al*., 1977).

The research group of the PHRI extended the study to investigate the behavior of improved ground (Terashi and Tanaka, 1981a, 1983; Terashi *et al*., 1983a, 1985). During this period in the early 1980s, the Japanese Geotechnical Society established a technical committee to compile the State of the Art of the deep mixing method and its essence was reported in the monthly journal of the Society.

For the researches on the machinery development, the equipment (Mark I to Mark III) was developed at the PHRI with the collaboration of Toho Chika Koki Co. Ltd. The first field test was done with the Mark II machine, which was only 2 m high. The first trial on the sea was done near-shore at Haneda Airport with the Mark III as shown in Figure 2, which was capable of improving the sea bottom sediment up to 10 m from the sea water level. The basic mechanism of the equipment was established by these trials. Finally the Mark IV machine was manufactured by Kobe Steel Co. Ltd. and a marine trial test was done by the PHRI near-shore at Nishinomiya to establish the construction control procedure. Stimulated by these activities in the development of the new technique, a number of Japanese contractors started their own research and development of this technique in the middle 1970s.

Figure 2 First full-scale test at offshore Haneda in the early 1970s (courtesy of Port and Airport Research Institute)

As granular quicklime or powdered hydrated lime was used as a binder in these initial development stages, the method was named the "Deep Lime Mixing (DLM)." The first contractor who put the DLM into practice was Fudo Construction Co. Ltd. The very first application was the use of the Mark IV machine to improve reclaimed soft alluvial clay in Chiba prefecture in 1974. In the five years before 1978, the DLM was practiced at 21 construction sites including two marine works.

In an effort to improve the uniformity of stabilized soil, cement mortar and cement slurry quickly replaced granular quicklime. The PHRI, Kawasaki Steel Corp. and Fudo Construction Co. Ltd. developed in corporation the deep mixing method with cement mortar as a binder in 1974, which was named the “Clay Mixing Consolidation Method (CMC).” The PHRI also developed the method with cement slurry as a binder in 1975 together with Takenaka Civil Engineering & Construction Co., Ltd. The deep mixing method using binder slurry is now called the wet method of deep mixing. These developments encouraged many marine contractors to develop their own method and machine in 1975 to 1977. In 1976, the Second District Port and Harbour Construction Bureau, the Ministry of Transport carried out a large scale exp eriment on the sea at the Daikoku pier in Yokohama Port, where the properties of the in-situ stabilized soil, the reliability of overlapped portion, construction ability were confirmed.

A research group at the Public Works Research Institute of the Japanese Ministry of Construction started studies to develop a similar technique from the late 1970s to the early 1980s, inviting staffs of the PHRI to take part as advisory committee members. The technique developed was called the "Dry Jet Mixing (DJM) Method" in which dry powdered cement or lime was used as a binder instead of binder-water slurry. This is now called the dry method of deep mixing.

Since a variety of equipment was established and standard design procedures became available, the application of the deep mixing method has exploded. The total volume of stabilized soil by the deep mixing method from 1977 to 2010 reached 72.3 million m^3 for the wet method and 32.1 million m^3 for the dry method.

Until the end of 1980s, the deep mixing method has been developed and practiced only in Japan and Nordic countries with a few exceptions. In the 1990s, the deep mixing method gained popularity also in the USA and central Europe. The first international specialty conference on deep mixing was co-organized by the Japanese Geotechnical Society and the International Society of Soil Mechanics and Geotechnical Engineering TC-17 in 1996 in Tokyo. The 1996 Tokyo Conference was followed by a series of specialty conferences/symposia in 1999 Stockholm, 2000 Helsinki, 2002 Tokyo, 2003 New Orleans and 2005 Stockholm. Along with these international forums, CEN TC288/WG10 started drafting the European standard of the execution and execution control of deep mixing in 2000. The WG 10 comprising delegates from 9 European countries invited international experts from Japan and the USA to take part in their activity and completed an international standard. Recently, the International Symposium on Deep Mixing and Admixture Stabilization, OKINAWA 2009, was held in Okinawa, Japan, which was a continuation of the tradition of deep mixing community but expanded the scope to cover similar admixture stabilization techniques. Now, the latest information on equipment, material properties, case records, design procedure, quality control (QC) and quality assurance (QA) have been updated and shared by international deep mixing community by conducting a series of international specialty meetings.

4. PATTERNS AND PURPOSES

4.1 Improvement Pattern

Since 1970s, the mechanical deep mixing method (DMM) has frequently been applied to the improvement of soft clays, organic soils and sandy soils for various purposes and in various ground conditions in on-land and marine constructions. A round column of stabilized soil is produced by a single stroke (penetration and withdrawal) of one-shaft deep mixing machine. As deep mixing machine in general has two to eight mixing shafts and blades in Japan, stabilized soil produced by a single stroke consists of several round columns partially overlapped each other. Stabilized soil columns are installed by a variety of patterns; block, wall, grid or group of individual columns. Figure 3 illustrates the typical improvement patterns.

4.2 Improvement Purposes

Figure 4 shows typical improvement purposes of the DMM in Japan for clayey soils and sandy soils. Applications to clayey and organic soils include increasing bearing capacity, reducing settlement, increasing passive earth pressure, reducing active earth pressure and increasing horizontal resistance of pile and sheet wall. Applications to sandy ground, on the other hand, include increasing bearing capacity, reducing settlement and preventing liquefaction. In on-land constructions, the deep mixing method has been applied to embankments, oil tanks, and building foundations, while the deep mixing method has been applied to breakwaters, sea revetments and piers in marine constructions. Other than those exemplified in the figure, the deep mixing is also applied for seepage shutoff, vibration and displacement barrier and immobilization of contaminated soil.

The DMM has been applied to sandy ground as a countermeasure of liquefaction. The block type improvement expects to increase liquefaction potential by stabilizing whole liquefiable soil. The grid type improvement, in the other hand, has adopted in some cases to increase liquefaction potential of unstabilized soil left within the grid where its shear deformation during seismic motion is reduced by the confinement effect of the grid walls. The high applicability of the method was demonstrated in the 1995 Hyogoken-Nambu Earthquake and the 2011 Tohoku Earthquake, where negligible damage took place on the piles installed in the untreated soil grid walls (Tokimatu *et al.*, 1996; Uchida *et al*. 2012).

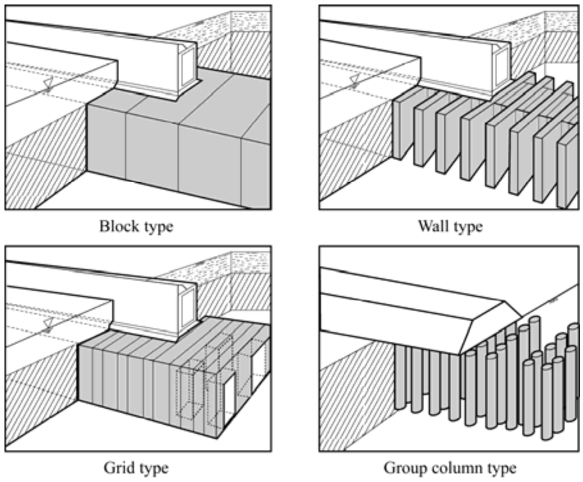


Figure 3 Improvement patterns of deep mixing improved ground

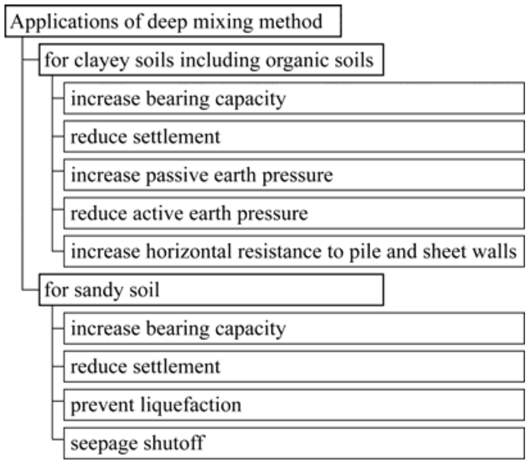


Figure 4 Improvement purposes

5. PERFORMANCE OF IMPROVED GROUND IN THE 2011 TOHOKU EARTHQUAKE

5.1 Introduction

The 2011 earthquake off the Pacific coast of Tohoku was a magnitude 9.0 (Mw) undersea mega thrust earthquake off the coast of Japan that occurred on 11 March 2011. It was the most powerful known earthquake ever to have hit Japan, and one of the five most powerful earthquakes in the world since modern record-keeping began in 1900. The earthquake resulted in a major tsunami that brought destruction along the Pacific coastline of Japan and resulted in the loss of thousands of lives and devastated entire towns. The degree and extent of damage caused by the earthquake and resulting tsunami were enormous, with most of the damage being caused by the tsunami. The aftermath of the 2011 Tohoku earthquake and tsunami included both a humanitarian crisis and massive economic impacts. The tsunami created over 300,000 refugees in the Tohoku region.

The Cement Deep Mixing Association, the Dry Jet Mixing Association and Chemical Grout Co. Ltd. conducted field surveys in the Tohoku and Kanto areas to investigate the performance of the improved grounds by the deep mixing. As the summary of the survey was already presented (Kitazume, 2012), the performances of the CDM and the DJM methods are briefly introduced. Table 2 summarizes the number of survey for the wet and dry methods of deep mixing. Though a few slight deformations were found in some improved grounds, as a whole no serious deformation and damage was found in the improved grounds and superstructures even they were subjected to quite large seismic forces. It can be concluded that the soil stabilization by deep mixing guarantees the high performance and high applicability for mitigating damages due to earthquake. The two examples of the field survey are briefly introduced here.

5.2 River Embankment Improved by the CDM Method

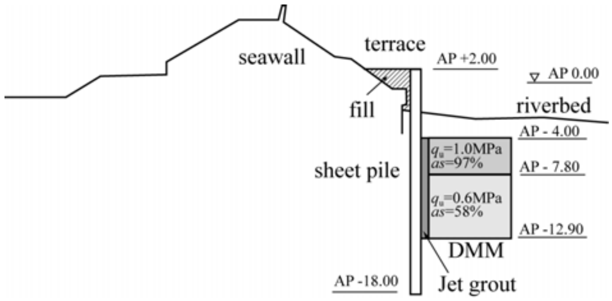
A part of the river embankment at the Naka River, Saitama Prefecture, was improved by the CDM method. The steel sheet pile wall installed at the front of the river embankment was improved by the CDM for increasing the horizontal resistance of the wall and stability of embankment. The jet grouting technique was also applied between the wall and the CDM ground to increase the lateral resistance. The CDM improved ground had about 7.0 m in width and 8.9 m in height, and where the design strength, q_{uck} and the improvement area ratio, a_s were 1.0 MN/m^2 and 0.97 for the upper part, 0.6 M N/m^2 and 0.58 for the lower part, as shown in Figure 5(a). The improvement execution was carried out in 2005 by the on-land type machine installed on the small verge.

The survey after the earthquake revealed that no damage was found in the embankment and the improved ground, even they were subjected to the large ground motion of the seismic force of 5.0 upper in Japanese Magnitude-Shindo (seismic intensity scale) as shown in Figure 5(b). In contrast to the improved ground, damage at river embankment without any ground improvement was found as shown in Figure 5(c).

5.3 River Embankment Improved by the DJM Method

The foundation for the river embankment in Chiba Prefecture was improved by the DJM method as shown in Figures 6(a) and 6(b), where the width and height of the grid type improved ground were 21.0 m and 21.0 m respectively. The improvement area ratio and the design strength were 0.506 and q_{uck} of 600 kN/m^2 respectively.

The survey after the earthquake revealed that no damage was found in the embankment and the improved ground, as shown in Figure 6(c).



(a) Cross section of the Naka River embankment

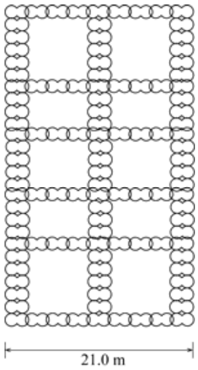
(b) River embankment at the Naka River

(c) River embankment without improvement

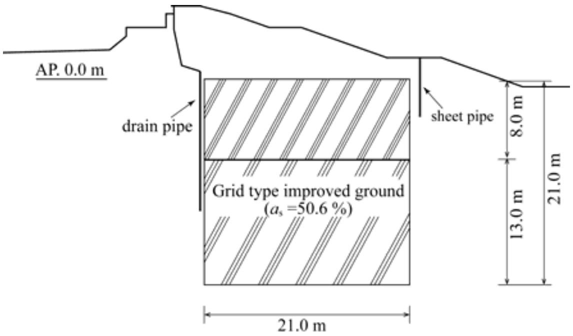
Figure 5 Comparison of the CDM improved ground and unimproved ground

Table 2 Summary of the survey

	Ao-mori	Iwate	Akita	Ya-maga-ta	Miya-gi	Fuku-shima	Ibara-gi	Chiba	Sai-tama	Tokyo	Kana-gawa	Total
wet method												
no. of projects	28	17	23	21	38	10	77	74	73	302	152	815
no. of surveys	15	9	8	9	27	2	27	28	37	153	85	400
dry method												
no. of projects	12	4	-	-	19	3	33	49	3	-	-	123
no. of surveys	8	2	-	-	14	1	21	28	3	-	-	77



(a) Cross section of the DJM improved ground



(b) Plan section of the DJM improved ground

(c) River embankment after earthquake

Figure 6 DJM improved ground for the river embankment in Chiba Prefecture

6. WORK FLOW FOR DM PROJECT

6.1 Work Flow of Deep Mixing

Figure 7 shows the work flow common to a project involving deep mixing (Terashi, 2003). The sequence of work items in the flow may change from a project to another depending on such factors as the size and complexity of the project, the variability of the subsurface conditions, and the anticipated difficulty of deep mixing at the project site. The role of the geotechnical design is to determine, based on the design parameters, the size of improved zone, installation depth and installation pattern so that the improved ground may satisfy the performance criteria of the superstructure. The currently available geotechnical design procedure is different for different column installation pattern. This is an iterative process and the engineer has to change the factors mentioned above until the appropriate solution is reached. The geotechnical designer should establish design parameters and required level of accuracy of installation considering the capability of the current deep mixing technologies.

The role of the process design is to determine the construction control values to realize the quality of improved ground specified by geotechnical design. Specifications may include not only the strength and uniformity of in-situ stabilized soil columns but also the accuracy of installation in order to guarantee the location, depth, stable contact with bearing layer and reliable overlap of columns. The process design is often made possible by the field trial installation using the locally available equipment and materials.

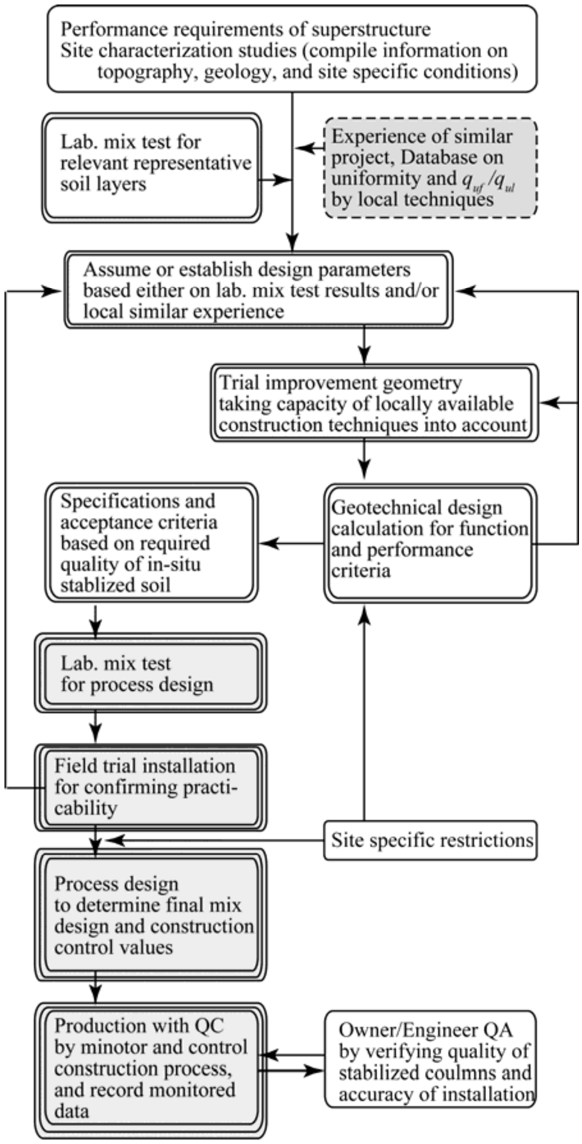


Figure 7 Work flow for the project involving deep mixing (Terashi, 2003)

The laboratory mix test is often carried out as a bench scale test to determine whether the soft soils at the project site are suitable for deep mixing. The strength of stabilized soils can be controlled by the amount of binder. However, the cost and the capability of the locally available deep mixing machines may restrict the upper limit for the quantity of binder. The properties and uniformity of the in-situ stabilized soil columns are influenced by many factors, among which the capability of deep mix machine and its operational conditions are important.

6.2 Geotechnical Design

The technical standard for the geotechnical design of improved ground by deep mixing as a foundation of port facilities such as breakwater or revetment by block type and wall type column installation patterns was first established in 1989 by the Ministry of Transport. When the deep mixing method expanded its application to various structures, several design standards or design guides have been tailored for specific structures by respective organizations which oversee them. The Public Works Research Center published the design method and commentaries of the group column type improved ground for embankment support in 1999. For applications to building foundation, the Building Center of Japan proposed the Design and Quality Control Guideline of Improved Ground for Building in 1997. The guideline was revised and authorized by Architectural Institute of Japan in 2006 (Architectural Institute of Japan, 2006). For applications to oil tank foundation, Fire and Disaster Management Agency gave the official notification on the design procedure for tank foundation in 1995, in which the design procedure of the deep mixing method was specified (Fire and Disaster Management Agency, 1995). The Ministry of Construction proposed a draft design method for liquefaction mitigation by the grid type of improvement (Ministry of Construction, 1999). These design procedures are not identical due to different performance and functional requirements specific to the type of structures.

6.3 Process Design

The process design is to determine the binder type, binder content, construction procedure, construction control items and construction control values in order to realize the required quality of in-situ treated soil (such as strength and uniformity) and to determine the construction procedure to realize the location, depth, contact with bearing layer, and reliable overlap of columns to the level of accuracy that the geotechnical design requires. Laboratory mix test and field trial installation are often carried out for the process design. Deep mixing contractor is al-so expected to cooperate the owner's quality assurance and verification. Results of verification testing together with the laboratory test results will be accumulated to improve the local database.

7. DESIGN PROCEDURE

7.1 Group Column Type Improved Ground

7.1.1 Introduction

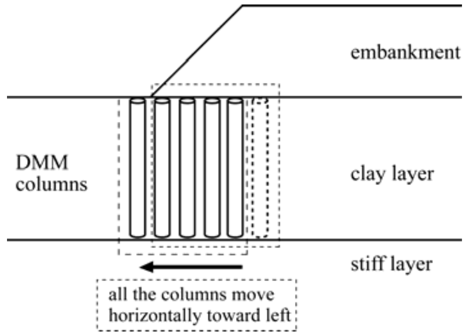
Nevertheless, the group columns are preferred even for the stability due to the simplicity in construction and cost and time saving. The design method for the group column type improved ground was proposed by the Public Works Research Center in 1999, and revised in 2004 (Public Works Research Center, 2004). In this section, the group column type improved ground beneath an embankment is exemplified, where the two dimensional condition is assumed. This section basically introduces the design methodology established by Public Works Research Center (Public Works Research Center, 2004).

7.1.2 Basic concept

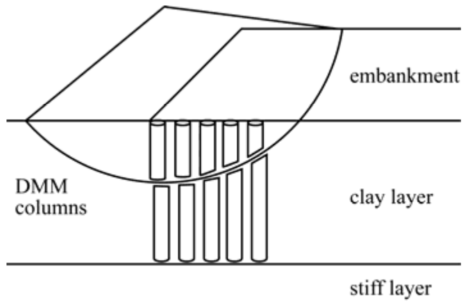
In the PWRC design, the group column type improved ground is considered to be a sort of composite ground with an average strength of stabilized soil columns and unstabilized soil between them. In the design, two stabilities are evaluated: external and internal stabilities. The external stability examines the possibility of sliding failure of improved ground, in which the stabilized soil columns and the unstabilized soil between them moves horizontally as shown in Figure 8(a). For the internal stability, the possibility of column failure is evaluated by slip circle analysis (see Figure 8(b)).

7.1.3 Design flow

The design flow for the group column type improved ground is shown in Figure 9. After determining the design conditions and dimensions of superstructure such as embankment, the dimensions of improved ground are assumed at the first step. The sliding stability analysis and slip circle analysis are conducted for the external and internal stabilities respectively. The horizontal displacement of the improved ground is examined in many cases. The bearing capacity and ground settlement are examined finally, and the details of the improved ground such as strength and dimensions are determined.



(a) External stability of improved ground



(b) Internal stability of improved ground

Figure 8 Failure pattern assumed in the PWRC design procedure

The width and depth of improved ground, improvement area ratio and strength of stabilized soil column are determined by trial calculations. Trial values for the initial design calculation are established / assumed by considering similar case histories. The width of improvement is usually assumed as the width of embankment side slope for increasing slope stability. For the settlement reduction, stabilized soil columns are installed beneath the full height of embankment.

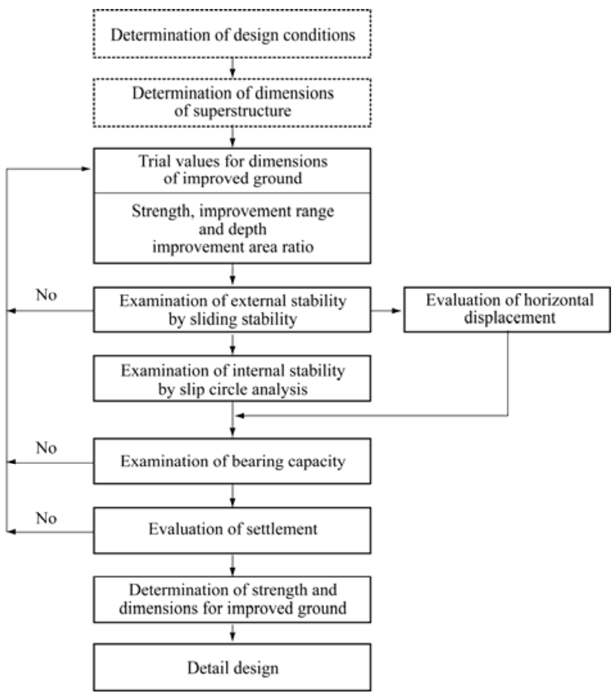


Figure 9 Design flow for group column type improvement
(after Public Works Research Center, 2004)

The depth of improvement is classified into two improvement types: fixed type and floating type improvements depending upon whether stabilized soil columns reach the stiff layer or not. It can be easily understood that the fixed type improvement is preferable from the viewpoints of increasing stability and reducing settlement. The depth of improvement is usually assumed as the bottom of soft ground, where the stabilized soil columns reach the stiff layer, the fix type improvement. In the case where the thickness of the soft ground is quite large, however, the floating type improvement is selected. As the appropriate range for the ratio of the width to the depth of improvement, 0.5 to 1.0 is recommended based on the accumulated experiences.

The improvement area ratio, a_s is represented as the ratio of the sectional area of stabilized soil column to the ground occupied by a single column. The improvement area ratio, a_s of 0.3 to 0.7 is usually adopted for foundation of embankment.

The design unconfined compressive strength of stabilized soil column, q_{uck} can be assumed at first by Equation (1) with the safety factor of 1.0 to 1.2. This equation means that the strength of stabilized soil column should be higher than the embankment load on the area occupied by the column. As explained later, the strength of stabilized soil column, however, is recommended to be 200 to 1,000 kN/m^2 by considering successful case histories.

$$q_{\text{uck}} \geq Fs \cdot \frac{\gamma_{\text{e}} \cdot H_{\text{e}}}{a_s} \tag{1}$$

where

- a_s : improvement area ratio
- Fs : safety factor
- H_{e} : height of embankment (m)
- q_{uck} : design unconfined compressive strength of stabilized soil (kN/m^2)
- γ_{e} : unit weight of embankment (kN/m^3)

7.1.4 Examination of external stability

For the external stability, the sliding failure of the improved ground is examined to determine the width and thickness of improved ground. In the design, the stability is evaluated based on the force equilibrium acting on both sides of the improved ground (Figure 10). The safety factor against sliding failure is calculated by Equation (2). In the calculation, the width and thickness of improved ground (mainly the width) are changed to assure the allowable magnitude of Fs_s which is usually 1.3 for static condition.

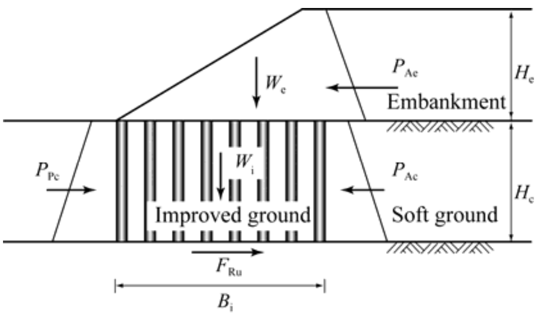


Figure 10 External force conditions for sliding failure analysis

$$Fs_{\text{s}} = \frac{P_{\text{Pc}} + F_{\text{Ri}}}{P_{\text{Ac}} + P_{\text{Ae}}} \tag{2}$$

where

- F_{Ri} : total shear force per unit length mobilized on bottom of improved ground (kN/m)
- Fs_{s} : safety factor against sliding failure of improved ground
- P_{Ac} : total static active force per unit length of soft ground (kN/m)
- P_{Ae} : total static active force per unit length of embankment (kN/m)
- P_{Pc} : total static passive force per unit length of soft ground (kN/m)

7.1.5 Examination of internal stability

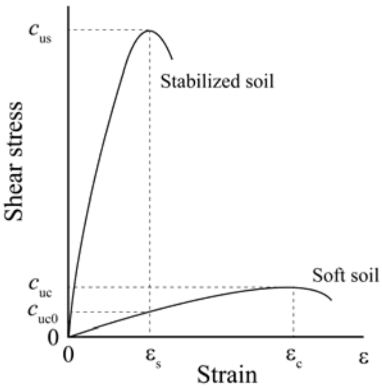
The internal stability analysis is evaluated by a slip circle analysis to determine the strength of stabilized soil column and the improvement area ratio. In the analysis, the composite ground consisting of stabilized soil columns and unstabilized soil is assumed to have an average strength defined by Equation (3). As the axial strain of stabilized soil at failure is in many cases considerably smaller than that of original soil, the shear strength of the original soil doesn't fully mobilize at the failure of the stabilized soil. This phenomenon is incorporated in Equation (3) by introducing the mobilization factor, k as shown in Figure 11(a).

The safety factor against slip circle failure, Fs_{sp} is calculated by the modified Fellenius analysis (see Figure 11(b)) with Equation (4). The allowable magnitude of safety factor of 1.3 is adopted for static condition in many cases.

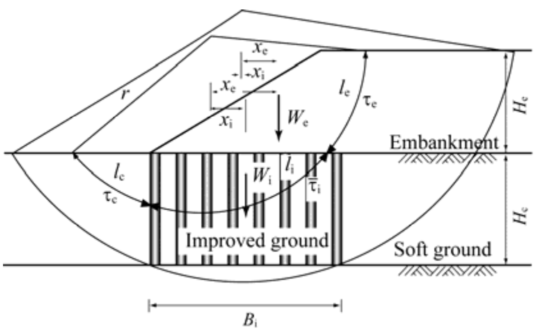
$$\bar{\tau}_{\text{i}} = a_{\text{s}} \cdot c_{\text{us}} + (1 - a_{\text{s}}) \cdot k \cdot c_{\text{uu}} \qquad k = \frac{c_{\text{u0}}}{c_{\text{uu}}} \tag{3}$$

where

- a_s : improvement area ratio
- c_{uu} : undrained shear strength of soft soil (kN/m^2)
- c_{u0} : undrained shear strength of soft soil mobilized at the peak shear strength of stabilized soil (kN/m^2)
- c_{us} : undrained shear strength of stabilized soil (kN/m^2)
- k : mobilization factor of soil strength
- τ_{i} : average shear strength of improved ground (kN/m^2)



(a) Illustration of stress and strain curves



(b) Slip circle analysis

Figure 11 Slip circle analysis

$$Fs_{\mathrm{sp}} = \frac{r \cdot \left(\tau_{\mathrm{c}} \cdot l_{\mathrm{c}} + \overline{\tau_{\mathrm{i}}} \cdot l_{\mathrm{i}} + \tau_{\mathrm{e}} \cdot l_{\mathrm{e}}\right)}{W_{\mathrm{e}} \cdot x_{\mathrm{e}}} \tag{4}$$

where

- Fs_{sp} : safety factor against slip circle failure
- l_{c} : length of circular arc in soft ground (m)
- l_{e} : length of circular arc in embankment (m)
- l_{i} : length of circular arc in improved ground (m)
- r : radius of slip circle (m)
- W_{e} : weight per unit length of embankment (kN/m)
- x_{e} : horizontal distance of weight of embankment from center of slip circle (m)
- τ_{c} : shear strength of soft ground (kN/m^2)
- τ_{e} : shear strength of embankment (kN/m^2)
- $\overline{\tau_{\mathrm{i}}}$: average shear strength of improved ground (kN/m^2)

Equation (4) often leads to misunderstanding that the improved ground having high strength of stabilized soil column and low improvement area ratio can be an alternative to low strength and large improvement area ratio to assure the required safety factor. The past experiences, however, have revealed that such alternative is not suitable because the composite ground concept can't be assured. The improvement area ratio of improved ground and the strength of stabilized soil column should be larger than 0.3 and ranging 500 to 1,000 kN/m^2 respectively in order to assure the composite ground concept.

7.1.6 Examination of horizontal displacement

The improved ground consisting of stabilized soil columns and surrounding soil may show horizontal and/or rotational displacement due to the weight of embankment and the earth pressures acting on the improved ground. When the improvement purpose includes the reduction of horizontal displacement that may give adverse influence on nearby existing structures, the examination of horizontal displacement is necessary. The PWRC recommends the use of two dimensional finite element analysis. Also recommended is the rough estimation of the horizontal displacement via the magnitude of minimum safety factor obtained by the slip circle analysis.

7.1.7 Examination of bearing capacity

The weight of embankment tends to concentrate on the stiff stabilized soil columns. The bearing capacity of the stiff layer at the bottom of the improved ground should be then examined. The PWRC design procedure doesn't specify any particular bearing capacity formula, but left it to the other design standards established by various organizations for specific facilities, such as road, railway, port facility and building.

7.1.8 Examination of settlement

In the settlement calculation for the fix type improved ground, it is usually assumed that the stabilized soil columns and the surrounding ground settle uniformly as illustrated in Figure 12, where the stress concentration effect is incorporated. This assumption has also been applied to flexible loading condition such as embankment. The final consolidation settlement of improved ground, S, is calculated by multiplying the final consolidation settlement of the original ground without improvement, S_{c} and a settlement reduction factor, β, as formulated by Equation (5).

The final consolidation settlement of the original ground is usually calculated by the Terzaghi's consolidation theory. In the case where the original ground consists of multiple layers, the settlement should be calculated as the sum up of the compressive deformations in each layer. The settlement reduction factor, β is derived by incorporating the stress concentration effect of the stabilized soil columns. The stress concentration ratio, n, can be calculated by a ratio of the coefficient of volume compressibility of the stabilized soil, m_{vs} and that of the unstabilized soil (original soil), m_{vc} as Equation (6).

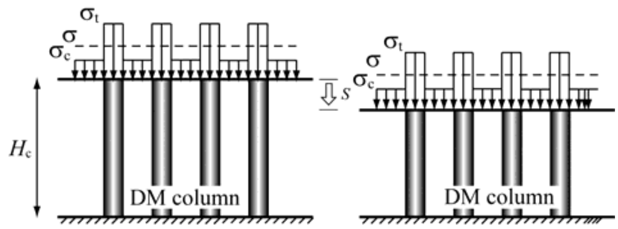


Figure 12 Calculation of consolidation settlement

$$S = \beta \cdot S_{\mathrm{c}}$$
$$\beta = \frac{1}{1 + (n-1) \cdot a_s} \tag{5}$$

$$n = \frac{\sigma_{\mathrm{s}}}{\sigma_{\mathrm{c}}} = \frac{m_{\mathrm{vc}}}{m_{\mathrm{vs}}} \tag{6}$$

where

- a_s : improvement area ratio
- n : stress concentration ratio ($\sigma_{\mathrm{s}}/\sigma_{\mathrm{c}}$)
- S : consolidation settlement of improved ground (m)
- S_{c} : consolidation settlement of soft ground without improvement (m)
- β : settlement reduction factor
- σ_{c} : vertical stress acting on soft ground between stabilized soil columns (kN/m^2)
- σ_{s} : vertical stress acting on stabilized soil columns (kN/m^2)
- m_{vc} : coefficient of volume compressibility of unstabilized soil (m^2/kN)
- m_{vs} : coefficient of volume compressibility of stabilized soil (m^2/kN)

In the case of the floating type improved ground, where a compressible layer is overlain by the improved ground, the ground settlement is calculated as the sum up of the settlement of improvement portion and that of unimproved portion. As the PWRC design procedure doesn't specify any design procedure, the design standard specified by the Building Center of Japan can be referred (The Building Center of Japan, 1997).

There have been some discussions on the permeability of stabilized soil (Terashi and Tanaka, 1981a, 1981b; Åhnberg, 2003) and whether the stabilized soil column can function as drainage like vertical drain method or not. The PWRC design standard doesn't specify the design procedure of the rate of consolidation settlement. However, as the accumulated data in Japan have revealed that the permeability of stabilized soil is lower than that of the original soil, it is usually assumed in Japan that the stabilized soil column doesn't function as drainage. Therefore the rate of consolidation settlement is usually calculated by similar manner of the Terzaghi's one dimensional theory with disregarding the stabilized soil columns.

7.2 Block Type and Wall Type Improved grounds

7.2.1 Introduction

The technical standard for the geotechnical design of improved ground by deep mixing as a foundation of port facilities such as breakwater or revetment by block type and wall type column installation patterns was first established in 1989 by the Ministry of Transport. In 2007, the design standard of deep mixing improved ground for port facilities was fully revised in which the reliability design concept was adopted. In the revised design method, the average and variation of soil parameters and external forces are incorporated by partial factors in the performance verifications (Ministry of Land, Infrastructure, Transport and Tourism, 2007). The standard and commentaries were published by the Ports and Harbours Association of Japan for the Japanese version (The Ports and Harbours Association of Japan, 2007) and by the Overseas Coastal Area Development Institute of Japan for the English version (The Overseas Coastal Area Development Institute of Japan, 2009). Here the design standard specified by the Ministry is briefly introduced, where the caisson type quay wall on the block type improved ground is shown as an example. The background of the standard and details on the partial safety factors are presented by Kitazume and Nagao (2007).

7.2.2 Basic concept

In the design method for port facilities, the stabilized soil of block or wall is not considered to be a part of ground, but rather to be a rigid structural member buried in a ground to transfer external forces to a reliable stratum. The average and variation of soil parameters and external forces are incorporated by partial factors in the performance verifications.

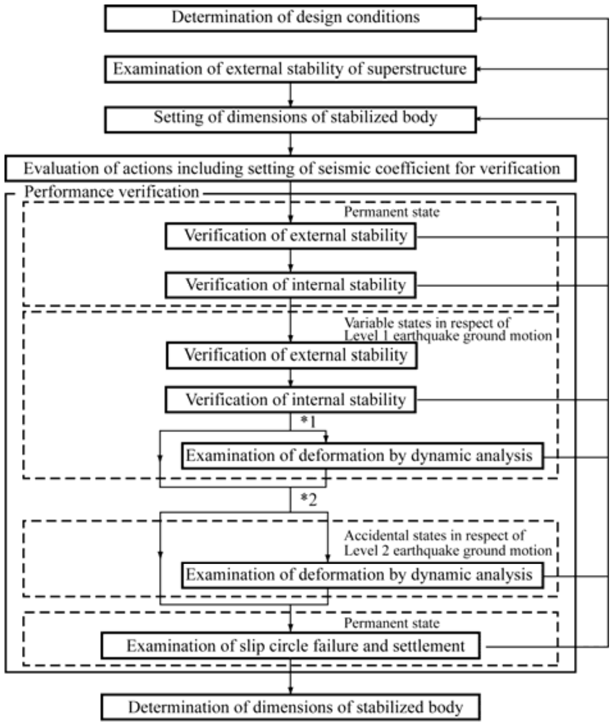
The Hyogoken-Nambu earthquake caused serious damages to many kinds of infrastructures and required to revise the seismic designs. The Japan Society of Civil Engineers proposed a new design concept for civil engineering infrastructures, in which seismic design of infrastructures should be evaluated under the Level 1 and Level 2 earthquake ground motions. The design assumes the Level 1 earthquake has a similar magnitude to those targeted in the previous design, which is estimated to take place once or twice in the life span of infrastructure. The Level 2 earthquake, on the other hand, is categorized into huge earthquake like the Hyogoken-Nambu earthquake. Its magnitude should be estimated by identifying the fault line and mechanism of anticipated earthquakes. Any infrastructures should be assured the seismic stability in the Level 1 earthquake ground motion. For the level 2 earthquake ground motion, any infrastructures should be assured the sustainability incorporating their importance.

The performance verification of variable states in respect of the Level l earthquake ground motion can be conducted, equivalent to gravity type quay walls, by either a simplified method or by a detailed method (nonlinear seismic response analysis considering dynamic interaction of the ground and structures). Examination of accidental states in respect of the Level 2 earthquake ground motion may also be necessary depending on the performance requirements of facilities.

7.2.3 Design flow

The design flow for the block type improved ground of port facilities is shown in Figure 13. The design concept is, for the sake of simplicity, derived by analogy with the design procedure for a gravity type structure such as a retaining structure.

The first step is evaluation of actions including setting of seismic coefficient for verification. The second step of the procedure is examination of external stability of superstructure to assure the superstructure and improved ground can behave as an unit. The third step is verification in permanent state, which includes verification of “external stability” and “internal stability” of improved ground. In the verification of the external stability, sliding failure, overturning failure and bearing capacity of the improved ground are evaluated. In the verification of the internal stability, the induced stresses due to the external forces are calculated and confirmed to be lower than the allowable values. The wall type improved ground is also examined for extrusion failure, where unstabilized soil between the long walls might be squeezed out. The fourth step is verification in the Level 1 earthquake ground motion, which includes verification of “external stability” and “internal stability” of improved ground. In some cases, the same verification is required for accidental state in respect of the Level 2 earthquake ground motion. Then, slip circle failure and settlement of improved ground are examined.



*1: When necessary, examination of deformation by dynamic analysis can be performed for Level 1 earthquake ground motion. In cases where the width of the improved subsoil is smaller than the width of the foundation mound, it is preferable to conduct an examination of deformation by dynamic analysis.

*2: Depending on the performance requirements of the main body, examination for Level 2 earthquake ground motion shall be performed.

Figure 13 Flow of the current design procedure
(The Ports and Harbours Association of Japan, 2007)

7.2.4 Examination of seismic coefficient for verification

The seismic coefficient of the Level 1 performance verification for superstructure on the DM improved ground can be obtained by Equation (7), which incorporates the allowable displacement of superstructure. The allowable displacement is specified in the standard depending on the type of structure, but should be specified depending upon its type and importance. In the case of gravity type quay wall, the D_a value of 100 mm is specified. The magnitude of the modified maximum seismic acceleration, α_c is obtained by seismic response analyses incorporating the maximum acceleration at bed rock, the ground conditions, and the time duration of earthquake (Kitazume and Nagao, 2007).

The seismic coefficient for the external forces acting on the improved ground, k_{h2k}, the seismic coefficient for dynamic earth pressures acting on the superstructure, k_{h2k}', and the seismic coefficient for dynamic earth pressures acting on the improved ground, k_{h3k}, can be calculated.

$$k_{h1_k} = 1.78 \cdot \left(\frac{D_a}{D_r} \right)^{-0.55} \cdot \frac{\alpha_c \cdot 0.64}{g} + 0.04 \tag{7}$$

where

D_a	: allowable displacement (mm)
D_r	: reference displacement (generally assumed to be 100 mm)
g	: gravity (= 9.8 m/s^2)
k_{h1k}	: seismic coefficient for superstructure
α_c	: modified maximum seismic acceleration (m/s^2)

7.2.5 Examination of external stability

In the "external stability analysis," three failure modes are examined for the assumed improved ground: sliding, overturning and bearing capacity failures. The design loads adopted in the external stability analysis are schematically shown in Figure 14. They include the active and passive earth pressures, surcharge and external forces acting on the boundary of improved ground, the mass forces generated by gravity, and the seismic inertia forces.

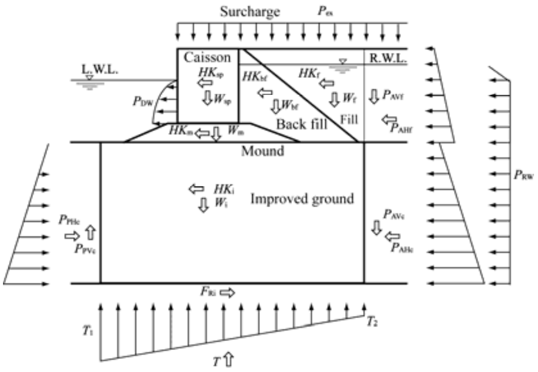


Figure 14 Schematic diagram of design loads (The Ports and Harbours Association of Japan, 2007)

In the calculation of sliding failure, it is assumed that the improved ground and the superstructure move horizontally on the stiff ground due to the unbalance of the earth pressures and /or the seismic inertia forces. The performance verification for the sliding failure is calculated by Equation (8), where the subscript *d* denotes the design value.

For permanent state:

$$P_{\mathrm{PHc_d}} + F_{\mathrm{Ri}_d} \geq \gamma_{\mathrm{a}} \cdot \gamma_{\mathrm{i}} \cdot \left(P_{\mathrm{AHc_d}} + P_{\mathrm{Rw_d}}\right) \tag{8a}$$

for variable states in respect of the Level 1 earthquake ground motion

$$P_{\mathrm{DPHc}_d} + F_{\mathrm{Ri}_d} \geq \gamma_a \cdot \gamma_i \cdot \left(P_{\mathrm{DAHc}_d} + P_{\mathrm{RW}_d} + P_{\mathrm{DW}_d} + \sum HK\right) \tag{8b}$$

where

- F_{Ri} : total shear force per unit length mobilized on bottom of improved ground (kN/m)
- P_{AHc} : horizontal component of total static active force per unit length (kN/m)
- P_{DAHc} : horizontal component of total dynamic active force per unit length (kN/m)
- P_{DPHc} : horizontal component of total dynamic passive force per unit length (kN/m)
- P_{Dw} : total dynamic water force per unit length (kN/m)
- P_{PHc} : horizontal component of total static passive force per unit length (kN/m)
- P_{Rw} : total residual water force per unit length (kN/m)
- ΣHK : sum of total seismic inertia force per unit length (kN/m)
- γ_{a} : structural analysis factor (generally assumed to be 1.0)
- γ_{i} : structural factor (generally assumed to be 1.0)

In the overturning failure, it is assumed that the improved ground and the superstructure rotate about the front bottom edge of the improved ground. The performance verification for the overturning failure is calculated by Equation (9), where the subscript *d* denotes the design value.

For permanent state:

$$\begin{aligned} & P_{\mathrm{PHc}_d} \cdot y_{\mathrm{PHc}} + P_{\mathrm{AVc}_d} \cdot x_{\mathrm{AVc}} + P_{\mathrm{SU}_d} \cdot x_{\mathrm{SU}} + W_{\mathrm{SP}_d} \cdot x_{\mathrm{SP}} + \sum W \cdot x \\ & \geq \gamma_a \cdot \gamma_i \cdot \left(P_{\mathrm{AHc}_d} \cdot y_{\mathrm{AHc}} + P_{\mathrm{RW}_d} \cdot y_{\mathrm{RW}}\right) \end{aligned} \tag{9a}$$

For variable states in respect of the Level 1 earthquake ground motion:

$$\begin{aligned} & P_{\mathrm{DPHc}_d} \cdot y_{\mathrm{DPHc}} + P_{\mathrm{DAVc}_d} \cdot x_{\mathrm{DAVc}} + P_{\mathrm{SU}_d} \cdot x_{\mathrm{SU}} + W_{\mathrm{SP}_d} \cdot x_{\mathrm{SP}} + \sum W \cdot x \\ & \geq \gamma_a \cdot \gamma_i \cdot \left(P_{\mathrm{DAHc}_d} \cdot y_{\mathrm{DAHc}} + P_{\mathrm{RW}_d} \cdot y_{\mathrm{RW}} + P_{\mathrm{DW}_d} \cdot y_{\mathrm{DW}} + \sum HK \cdot y\right) \end{aligned} \tag{9b}$$

where

- P_{AVc} : vertical component of total static active force per unit length (kN/m)
- P_{DAVc} : vertical component of total dynamic active force per unit length (kN/m)
- P_{su} : total surcharge force per unit length (kN/m)
- x_{AVc} : horizontal distance of vertical component of total static active force from bottom of improved ground (m)
- x_{DAVc} : horizontal distance of vertical component of total dynamic active force from bottom of improved ground (m)
- x_{sp} : horizontal distance of weight of superstructure from its edge (m)
- x_{su} : horizontal distance of total surcharge force from front edge of improved ground (m)
- y_{AHc} : vertical distance of horizontal component of total static active force from bottom of improved ground (m)
- y_{DAHc} : vertical distance of horizontal component of total dy namic active force from bottom of improved ground (m)
- y_{Dw} : vertical distance of total dynamic water force from bottom of improved ground (m)
- y_{DPHc} : vertical distance of horizontal component of total dy- namic passive force from bottom of improved ground (m)
- y_{Rw} : vertical distance of total residual water force from bot tom of improved ground (m)
- y_{PHc} : vertical distance of horizontal component of total static passive force from bottom of improved ground (m)
- W_{sp} : weight per unit length of superstructure (kN/m)
- ΣHKy : sum of total seismic inertia moment force per unit length (kN/m)
- ΣWx : sum of moment force per unit length (kN/m)
- γ_{i} : structural factor (generally assumed to be 1.0)
- γ_{a} : structural analysis factor

As the deep mixing improved ground is assumed as a buried structure in this design procedure, its bearing capacity is evaluated by the classical bearing capacity theory which can incorporate the effects of loading condition and embedded condition. In the design, the subgrade reactions at the front edge and the rear edge of the bottom of improved ground are calculated. The performance verification for the bearing capacity is calculated as Equation (10), while the bearing capacity of the improved ground is calculated by Equation (11) , where the subscript *d* denotes the design value.

$$\begin{aligned} t_1 &\leq q_{\mathrm{ar}_d} \\ t_2 &\leq q_{\mathrm{ar}_d} \end{aligned} \tag{10}$$

$$q_{\mathrm{ar}_d} = \gamma_R \left(\gamma_{\mathrm{b}_d} \cdot \frac{B_{\mathrm{i}}}{2} \cdot N_{\gamma_d} + c_{\mathrm{ub}} \cdot N_{\mathrm{c}_d} + q \cdot \left(N_{\mathrm{q}_d} - 1\right) \right) + q \tag{11}$$

where

- B_{i} : width of improved ground (m)
- c_{ub} : undrained shear strength of soil beneath improved ground (kN/m^2)
- q : effective overburden pressure at bottom of improved ground (kN/m^2)
- q_{ar} : bearing capacity (kN/m^2)
- γ : unit weight of soil beneath improved ground (kN/m^3)
- N_c : bearing capacity factor of soil beneath improved Ground

N_q : bearing capacity factor of soil beneath improved ground
N_γ : bearing capacity factor of soil beneath improved ground

7.2.6 Examination of internal stability

In the "internal stability analysis," the induced stresses in the improved ground are calculated based on the elastic theory. The shape and size of the improved ground are determined so that the induced stresses are lower than the allowable strengths of the stabilized soil. In the calculation, the stabilized soil is generally assumed to have uniform property for the sake of simplicity even it contains possibly weaker zones due to construction process such as overlap joints. The effect of the strength at the overlapping portion is taken into account when determining the allowable strengths of stabilized soil.

According to the accumulated experiences in the design, the internal stability evaluation at the two critical parts as shown in Figure 15 is considered sufficient as long as the shape of stabilized soil is within the experiences: (a) subgrade reactions at the front edge and rear edge of improved ground, and (b) average shear stress along vertical shear plane at the front edge of superstructure. For the former, the subgrade reactions at the front edge and rear edge of the improved ground, t_1 and t_2, should be smaller than the design value as shown in Equation (12) , where γ is the partial factor, and the subscript d denotesthe design value.

For permanent state:

$$f_{c_d} \geq \gamma_a \cdot \gamma_i \left(t_{1_d} - P_{\mathrm{PHC}_d}\right)$$
$$f_{c_d} \geq \gamma_a \cdot \gamma_i \left(t_{2_d} - P_{\mathrm{AHC}_d}\right) \tag{12a}$$

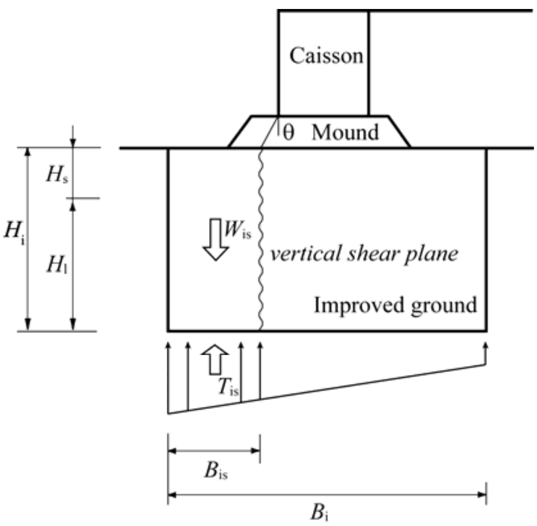


Figure 15 Internal stability of improved ground

For variable states in respect of the Level l earthquake ground motion

$$f_{c_d} \geq \gamma_a \cdot \gamma_i \left(t_{1_d} - P_{\mathrm{DPHC}_d}\right)$$
$$f_{c_d} \geq \gamma_a \cdot \gamma_i \left(t_{2_d} - P_{\mathrm{DAHC}_d}\right) \tag{12b}$$

where
f_c : design compressive strength (kN/m^2)
t_1 : subgrade reaction at front edge (kN/m^2)
t_2 : subgrade reaction at rear edge (kN/m^2)
γ_i : structural factor (generally assumed to be 1.0)
γ_a : structural analysis factor (generally assumed to be 1.0)

The average shear stress induced along the vertical shear plane at the front face of superstructure should satisfy the criteria as shown in Equations (13) , where γ is the partial factor, and the subscripts k and d denote the characteristic value and design value respectively. In the case where a mound underlies the superstructure, the stress distribution at the angle of around 30º can be taken into account to find the vertical shear plane.

$$f \leq f_{\mathrm{sh}_d} \tag{13a}$$

$$f = \gamma_a \cdot \gamma_i \cdot \frac{1}{H_\mathrm{i}}\left(\int_0^{B_\mathrm{is}} t_\mathrm{is}\,dt - W_{\mathrm{is}_d}\right) \tag{13b}$$

where
B_is : width of vertical shear plane from toe of improved ground (m)
f : average shear stress along vertical shear plane (kN/m^2)
f_sh : allowable shear strength of stabilized soil (kN/m^2)
H_i : height of improved ground (m)
t_is : reaction pressure at bottom of improved ground (kN/m^2)
W_is : weight per unit length of improved ground at part of B_is (kN/m)

Allowable strengths of stabilized soil:
The design strengths of stabilized soil are defined by Equations (13).

$$f_\mathrm{c} = \alpha \cdot \beta \cdot q_{\mathrm{uc}_d}$$
$$= \alpha \cdot \beta \cdot \gamma_{q_\mathrm{uc}} \cdot q_{\mathrm{uc}_k}$$
$$f_\mathrm{sh} = \frac{1}{2} f_\mathrm{c}$$
$$f_\mathrm{t} = 0.15 \cdot f_\mathrm{c} \leq 200 \text{ kN/m}^2 \tag{14}$$

where
f_c : design compressive strength of stabilized soil (kN/m^2)
f_sh : design shear strength of stabilized soil (kN/m^2)
f_t : design tensile strength of stabilized soil (kN/m^2)
q_uck : design unconfined compressive strength of stabilized soil (kN/m^2)
α : coefficient of effective width of stabilized soil column
β : reliability coefficient of overlapping

Slip circle analysis is carried out to evaluate the overall stability of the improved ground, the superstructure and the surrounding soil. As the strength of stabilized soil is very high value, a slip circle analysis passing through the improved ground is not necessary in many cases. In the case where sufficient bearing capacity is assured, slip circle analysis is not necessary in many cases.

7.2.7 Examination of immediate and long term settlements

After the optimum cross section of the improved ground is determined by the above procedures, the immediate and the long term settlements of the improved ground should be examined. Usually, the deformation of the stabilized soil itself can be negligible because of its high rigidity and large consolidation yield pressure. Therefore, the displacement of the improved ground is calculated as the deformation of the soft layers surrounding or beneath the stabilized soil. In the case of the fix type improvement where the stabilized soil reaches the stiff layer, the settlement can be assumed to be negligible. In the case of the floating type improvement, the consolidation settlement beneath the improved ground is calculated by the Terzaghi's one dimensional consolida-tion theory.

8. EXECUTION – EQUIPMENT, PROCEDURES AND CONTROL

8.1 Introduction

The techniques most commonly employed for in-situ deep mixing in Japan can be divided into three groups: mechanical mixing by vertical rotary shafts with mixing blades at the bottom end of each mixing shaft, high pressure injection mixing, and combination of the mechanical mixing and high pressure injection mixing. The various methods in these groups are classified in Table 1.

Here, the deep mixing equipment, construction procedure and quality control methods is introduced for the representative wet type deep mixing techniques in Japan, CDM. The other methods such as the dry method of deep mixing, DJM, the high pressure injection mixing, Jet Grouting, and the hybrid of mechanical mixing and high pressure injection mixing are referred in the literature (Kitazume and Terashi, 2013).

8.2 Wet Method of Deep Mixing for On-land Works

8.2.1 Machinery

For the wet method of deep mixing a variety of deep mixing machines are developed by deep mixing contractors to meet the purpose of improvement and applications and their specifications are quite variable. A system of the Cement Deep Mixing (CDM) method consists of a DM machine and a binder plant. The binder plant consists of a binder silo, water tank, binder-water mixer, agitator tank, pumping unit and control room. The CDM machine consists of a mixing tool and a crawler crane with a leader. The crawler cranes with a lifting capacity of 250 to 550 kN are often used as a base carrier. The CDM machine can be classified into four groups depending on their size of base carrier and the maximum stabilization depth.

The ordinary CDM machines for on-land works have two mixing shafts. The set of mixing shafts are suspended along the leader and laterally clamped at the top of mixing tool and the bottom of leader (Figure 16). The motor and gear box are installed on the top of the shafts. Binder slurry is supplied to each shaft by independent pumping unit to enable even delivery of binder slurry to each shaft. A swivel joint is installed at the top of each mixing shaft for binder slurry supply. The motor for driving mixing shafts is different for each group, two 45 kW motors for the 10 m class, two 50 to 60 kW motors for the 20 m class, two 75 to 90 kW motors for the 30 m class and two 90 kW motors for the 40 m class. The spacing of the mixing shafts are either 0.8, 1.0 or 1.1 m for the diameter of mixing blades of 1.0, 1.2 and 1.3 m respectively, to produce a stabilized soil element consisting of two partially overlapped round columns. The cross sectional area of a stabilized soil element ranges from 1.5 to 2.6 m^2.

Figure 16 Typical CDM machine for on-land work in operation

A double shafts machine usually has a bracing plate to keep the distance of two mixing shafts (see Figure 17). The plate is also expected to function to increase mixing degree by preventing the “entrained rotation phenomenon,” a condition in which disturbed soil adheres to and rotates with the mixing blade without efficient mixing of soil and binder. For a single shaft machine, a "free blade," an extra blade about 100 mm longer than the diameter of mixing blade, is installed when necessary, close to one of the mixing blades to prevent the "entrained rotation phenomenon." Two shafts of the double shaft machine rotate in the opposite direction, which increase the degree of mixing and also improve the stability of the machine.

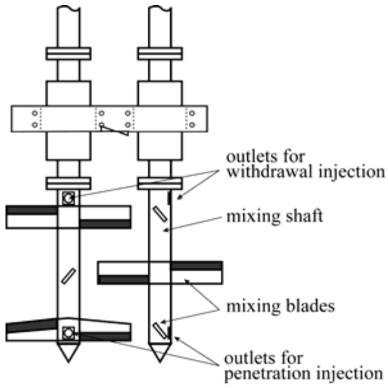


Figure 17 Typical mixing blades for the CDM method for on-land work

The mixing shaft is 267 mm circular shape. A duct with 50 mm in diameter is installed in the mixing shaft, through which binder slurry is supplied to the mixing blades. A stack of blades is installed at the bottom end of mixing shaft, which consists of excavation blade and mixing blades, as shown in Figure 18. The excavation blade is installed at the very end of mixing shaft, on which forks made by hard metal are fixed so that the machine can excavate and screw in a soil efficiently. The mixing blades at different levels are inter-sected at right angles each other. Two outlets of binder slurry are installed on the shafts at different levels close to the mixing blades, so that the outlets are not blocked by the soil. The upper outlet is used for withdrawal injection and the lower one is for penetration injection. The shape and the number of mixing blades have been developed to assure high mixing degree as much as possible, and now have various variations depending upon the contractors, as shown in Figure 18

A binder plant is prepared for producing and supplying binder slurry to the CDM machine. A silo of the maximum capacity of 30 ton in general is prepared for storage of binder. Binder slurry is usually manufactured by every 1 m^3 in a mixer of 1.5 m^3 capacity, and temporarily stored in an agitator of 2.0 to 3.0 m^3 in capacity. The water to binder ratio (W/C) of binder slurry is usually 60 to 100 %. The binder slurry thus manufactured is supplied to each mixing shaft of the CDM machine by the independent pump, where total of about 100 to 350 *l*/min. in volume is supplied to the machine by the help of pumping pressure of about 2.5 MN/m^2.

A control unit is installed in a control room in many cases, but in some cases on the CDM machine, where the binder condition, the amount of each material, the rotation speed of mixing blades, the penetration and withdrawal speeds of mixing shafts, *etc*. are continuously monitored, controlled, and recorded.

8.2.2 Construction procedure

Field preparation is carried out in accordance with the specific site conditions, which includes suitable access for plant and machinery, leveling of the working platform. Before actual operation, execution circumstances should be prepared to assure smooth execution and prevention of environmental impact.

(a) CDM method

(b) CDM Mega method

(c) CDM Column method

(d) CDM Land4 method

Figure 18 Various types of mixing shaft and blades for CDM method (courtesy of Cement Deep Mixing Method Association)

After setting the machine at the prescribed position, the mixing tool is penetrated into a ground while rotating the mixing shafts. There are two basic execution procedures depending on the injection sequence of binder: (a) injecting binder slurry during the penetration of mixing shafts and (b) injecting binder slurry during the withdrawal of mixing shafts. The location of the injection outlet is different for each injection method. For the penetration injection method, the injection outlets should locate at the lowest mixing blades, but they should be at the uppermost mixing blades for the withdrawal injection. The penetration injection is frequently applied to the CDM method for on-land work.

Ordinary execution process of the CDM method is shown in Figure 19, where binder slurry is injected during the penetration stage. During the penetration, the mixing blades are rotating to disaggregate and disturb the soil to reduce the strength of ground so as to make the mixing tools penetrate by their self-weight. The binder slurry is injected during penetration and mixed with the disaggregated soil. The mixing also continues in the withdrawal stage. The flow rate of binder slurry is kept constant while the penetration speed is controlled constant so as to assure the design amount of binder should be mixed. In the withdrawal stage, the direction of the mixing blade rotation is reversed and the binder is mixed with the soil again.

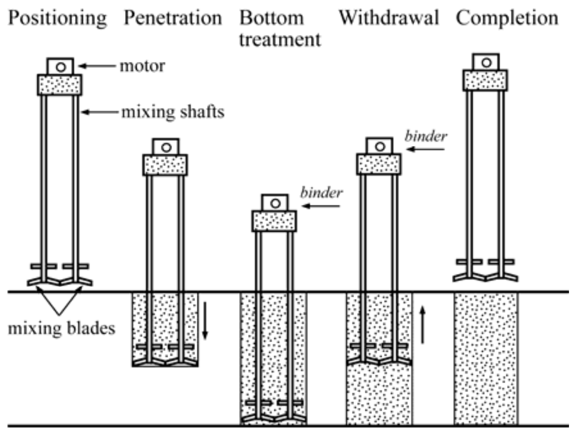


Figure 19 Execution process of the CDM method

The stabilized soil columns should reach a stiff layer sufficiently in the case of the fixed type improvement. In practical execution, rapid change in the penetration speed of mixing shaft, required torque and rotation speed of mixing blades are useful to detect whether the mixing blades have reached the stiff layer. When the mixing tool reached the stiff layer, the machine stays there for several minutes or goes up and down about one meter with continuing injection of binder slurry and mixing to assure sufficient contact of the column with the stiff layer.

Different operational parameters are used for the penetration injection and withdrawal injection in order to achieve the same level of mixing degree. The "blade rotation number" as defined by Equation (15) of about 350 is attained by the set of typical operational parameters both for penetration and withdrawal injection. This number is proposed to assure sufficient homogeneity of the stabilized soil column according to experience and research efforts.

For penetration injection:

$$T = \sum M \cdot \left(\frac{N_d}{V_d} + \frac{N_u}{V_u} \right) \tag{15a}$$

For withdrawal injection:

$$T = \sum M \cdot \left(\frac{N_u}{V_u} \right) \tag{15b}$$

Where

N_d : number of rotation of mixing blades during penetration (N/min)
N_u : number of rotation of mixing blades during withdrawal (N/min)
T : blade rotation number (N/m)
V_d : penetration speed of mixing blades (m/min)
V_u : withdrawal speed of mixing blades (m/min)
ΣM : total number of mixing blades

9. QC/QA FOR IMPROVED GROUND

9.1 Introduction

The quality of stabilized soil depends upon a number of factors including the type and condition of original soil, the type and amount of binder, and the execution process. The quality control and quality assurance (QC/QA) practice which focuses upon the quality of stabilized soil was originally established in Japan and Nordic countries and has been accepted worldwide for more than three decades. It comprises laboratory mix test, field trial test, monitoring and control of construction parameters during execution and the verification by measuring the engineering characteristics of stabilized soil either by unconfined compression tests on core samples or by sounding. Diversification of application, soil type, and execution system, together with the improved understanding on the behavior of improved ground necessitate our profession to review the current QC/QA practice.

The purpose of deep mixing is not only to manufacture a good quality stabilized soil but to create an improved ground which guarantees the performance of superstructure. The improved ground by the deep mixing method is a composite system comprising stabilized soil columns and original soils.

9.2 Process Design

The process design is to determine the binder type, binder content, construction procedure, construction control items and construction control values in order to realize the required quality of in-situ stabilized soil (such as strength and uniformity) and to determine the construction procedure to realize the location, depth, contact with bearing layer, and reliable overlap of columns to the level of accuracy that the geotechnical design requires. Laboratory mix test and field trial test are often carried out for the process design. Deep mixing contractor is also expected to co-operate the owner's quality assurance and verification. Results of verification testing together with the laboratory test results will be accumulated to improve the local database. Quality assurance of the deep mixing method to fulfill the requirements of geotechnical design cannot be achieved only by the process control (QC) during construction conducted by deep mixing contractor, but it should involve relevant activities that are carried out prior to, during and after the construction by all the parties involved in the deep mixing project. Usually the site investigation of original ground, for example, is not considered as a part of QA but it is underlined and classified as one of the important relevant activities. If the site investigation failed to identify the existence of problematic layer, the laboratory mix test would not be undertaken for the layer, which might result in insufficient process design (including QC/QA methods/procedures) and would cause difficulty in interpretation of the field trial stabilized soil columns and/or verification test of production columns.

Whatever the type of application and the function of stabilized soil columns, it is important to discuss the QC/QA procedures for the stabilized soil. The strength of stabilized soil is affected by many factors such as soil properties (natural water content, liquid limit, plastic limit, *pH*, organic matter content, grain size distribution and clay minerals), type and quantity of binder, mixing degree, and curing conditions. The effects of these factors are quite complex, making it difficult to directly determine field strength only by laboratory mix test.

The deep mixing machines must be simple and tough enough to endure severe working conditions. Mixing time in practice must be as short as possible for economic reasons. Hence, in-situ mixing conditions and curing conditions are quite different from the standard laboratory testing, and the strength of the in-situ stabilized soil column is usually different from that in the laboratory. The in-situ stabilized soil columns have relatively large strength variability even if the execution is done with the established mixing machine and with the best care. Average compressive strength, q_{ul} and the deviation of the laboratory specimen and the average strength, q_{uf} and the deviation of in-situ column are schematically shown in Figure 20. Usually the in-situ stabilized soil column has smaller average strength and larger strength deviation than those of the laboratory specimen. The design strength, q_{uck}, is derived from q_{uf} by incorporating the strength deviation as Equation (16). The target strength of the laboratory specimen should be determined by incorporating the strength difference and the strength deviation. When using statistical measures for quality control, the following relationship between field strength and the design standard strength must be formulated if the field strength of the improved soil is assumed to have a normal distribution curve.

$$\begin{aligned} q_{uck} &\leq \overline{q_{uf}} - K \cdot \sigma \\ \overline{q_{uf}} &= \lambda \cdot \overline{q_{ul}} \end{aligned} \tag{16}$$

where

K : coefficient
q_{uck} : design standard strength (kN/m^2)
$\overline{q_{uf}}$: average unconfined compressive strength of in-situ stabilized column (kN/m^2)
$\overline{q_{ul}}$: average unconfined compressive strength of laboratory stabilized soil (kN/m^2)
σ : standard deviation of the field strength (kN/m^2)
λ : ratio of q_{uf}/q_{ul}

To ensure the sufficient quality of the stabilized column, quality control and quality assurance is required before, during and after construction. For this purpose, quality control for the deep mixing method mainly consists of i) laboratory mix tests, ii) field trial test, iii) quality control during construction and iv) quality assurance after construction through laboratory test on core samples and pile head inspection.

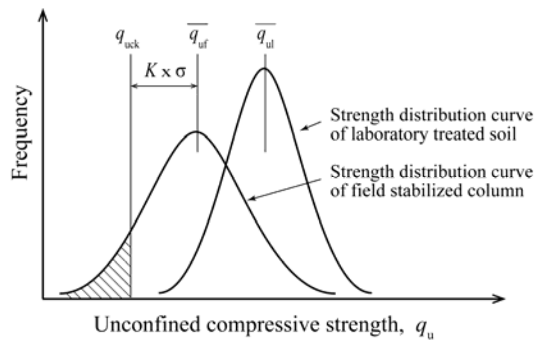


Figure 20 Field and laboratory strength of stabilized soil

9.3 Laboratory Mix Test

Laboratory mix test is an important pre-production QA which may be carried out in a different phase or phases of a project either for the geotechnical design or for the process design. Laboratory mix test is the responsibility of the owner/engineer if the deep mixing work is awarded with detailed specifications, but is the responsibility of the deep mixing contractor if the contract is awarded by performance basis.

Laboratory strength is influenced by many factors, such as mixing and molding conditions, curing condition, and testing conditions. To avoid the influence of these factors, the Japanese Geotechnical Society officially standardized the procedure in 1990, and made minor revisions in 2000 and 2009 (Japanese Geotechnical Society, 2009). Almost all laboratory tests for practical and research purposes follow this standard in Japan, which makes Japanese engineers rely upon test results obtained by different parties.

9.4 Field Trial Test

Field trial test is an important pre-production QA for deep mixing project especially when no comparable experience is available. It is recommended to conduct field trial test in advance in or adjacent to the construction site, in order to confirm the actual strength and uniformity in the real construction condition and determine the operational parameters and final mix design for production. The trial penetration of the deep mixing machine at the construction site without injecting the binder is a common practice in Japan to determine the process control value to confirm the end-bearing of columns to the stiff stratum if it is required. The change in the electric or hydraulic power consumption, change in torque and/or the change of penetration speed are measured during the trial installation to establish the construction control criteria for end bearing. Field trial installation for this purpose should be conducted in the vicinity of existing boring to compare with the known soil stratification.

9.5 Quality Control during Production

During production, stabilized soil columns must be installed to satisfy both the geometric layout and the quality of stabilized soil specified by the geotechnical design. Rig operator should locates, control, monitor and record geometric layout of each column (plan location, verticality and depth). When the termination depth is designated to ensure the reliable contact to the underlying stiff layer, rig operator should carefully identify the depth according to the construction control criteria established in the field trial test.

Quality control of the stabilized soil includes the binder storage, binder or binder slurry preparation, and control of mixing process. Storage and proportioning of binder, additives and mixing water are normally controlled, monitored and recorded at the plant placed in the construction site. Construction control parameters during column installation include the continuous monitoring of penetration and withdrawal speed, rotation speed, quantity of binder, water / binder ratio (for the wet method). The construction control values are predetermined by the process design considering the results of laboratory mix test, field trial test, and contractors' experience. During column installation, construction control values are controlled, monitored and displayed in the control room at the plant and/or cab of the mixing machine for the plant operator and rig operator to adjust the execution procedure when necessary. The mixing shaft and mixing tools are frequently observed for any possible defects during construction.

Reporting the recorded construction control parameters is an important QA during production. This is because the quality of stabilized soil column may be consistent if the construction process in a same project site is consistent.

The mixing degree mostly depends on the rotation speed of the mixing blade and penetration and withdrawal speeds of the shaft. In Japan, an index named "blade rotation number", T has been introduced to evaluate the mixing degree. This number means the total number of mixing blade passes during 1 m of shaft movement and is defined by the following equation for the penetration injection method and withdrawal injection method respectively. According to the accumulated researches and investigations, "blade rotation number" should be around 270 or larger to assure sufficient mixing degree for Japanese wet and dry methods, CDM and DJM.

To produce stabilized soil columns/elements that meet the design requirements on the quality and dimension, it is essential to control and monitor the quality of binder, geometric layout, and operational parameters such as amount of binder, rotation speed of mixing blades, shaft speed, *etc*. Figure 21 shows the operational parameters for the CDM method and items for geometric layout (Kitazume and Terashi, 2013). The verticality of the mixing tool is usually evaluated by the measurement of the verticality of leader, and is controlled within 1/200 to 1/100 in many cases. During production the monitoring data are fed back to the plant operator in the control room or the rig operator in the cab on the machine for precise construction. In practice, the rotation speed of mixing shafts is usually fixed. The flow rate of binder slurry is adjusted to the penetration or withdrawal speed by controlling the pumping pressure at the pumping units. The *W/C* ratio and density of binder slurry are controlled to the design value in the binder plant. The binder slurry should be used within about one hour after preparation to prevent the setting of binder before injection into the soil.

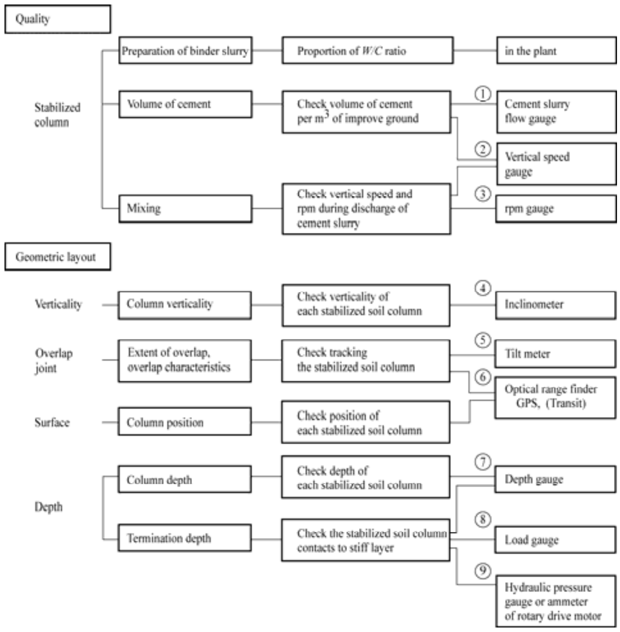


Figure 21 Operation monitoring for CDM method on-land works (Kitazume and Terashi, 2013)

9.6 Quality Verification

After the construction work, in-situ stabilized soil elements should be investigated in order to verify the design quality, such as continuity, uniformity, strength, permeability and dimension. In Japan, full depth coring and unconfined compression test on the core samples are most frequently conducted for verification. The number of core borings is dependent upon the number of stabilized soil elements in the project. In the case of on-land works, three core borings are generally conducted in the case where the total number of elements is less than 500. When the total number exceeds 500, one additional core boring is conducted for every further 250 elements. The continuity and uniformity of the stabilized soil column are confirmed by visual observation of the continuous core. Determination of the engineering properties of the stabilized soil is based on unconfined compressive strength on samples selected from the continuous core. The number of test depends upon the construction's condition and the soil properties. In general three core barrels are selected from three levels and three specimens are taken from each core barrel and subjected to unconfined compression test for each core boring.

The quality of the core sample primarily depends on the uniformity of stabilized soil. However, it further relies on the quality of boring machine, coring tool and the skill of workmen. If the coring is not properly conducted, low quality sample with some cracks can be obtained. A Denison type sampler, double tube core sampler or triple tube core sampler has been used for core sampling of stabi lized soil whose unconfined compressive strength ranges from 100 to 6,000 kN/m^2. It is recommended to use samplers of relatively large diameter such as 86 or 116 mm in order to take good quality samples. The quality of core sample is usually evaluated by visual inspection and/or the Rock Quality Designation (*RQD*) index. The *RQD* index measures the percentage of "good rock" within a borehole and provides the rock quality. The evaluation of the quality of the retrieved core in Japan varies from the subjective judgment such as good or bad by visual observation to the strict requirement of core recovery ratio of 100 % and the *RQD* value larger than 90 %.

The primarily used verification technique for the field strength is unconfined compression test on drilled core samples both for the wet and dry methods in Japan and the US. That for Nordic dry method is the column penetration test (Larsson, 2005). This difference in the preferred verification technique corresponds to the preferred field strength. Continuity of the stabilized soil column is verified by the visual observation and the core recovery ratio of core run in Japan and the USA, and by the column penetration or by the reverse column penetration in Nordic countries.

A variety of verification test procedures to examine the engineering characteristics of stabilized soil have been proposed (Hosoya *et al*., 1996; Larsson, 2005). However, actual practices rely upon traditional verification techniques such as the unconfined compression test on drilled core samples and/or the column penetration test.

10. CONCLUDING REMARKS

The Deep Mixing Method (DMM), a deep in-situ soil stabilization technique using cement and/or lime as a stabilizing agent, was developed in Japan and in the Nordic countries independently in 1970s. Due to its wide applicability and high improvement effect, the method has gained increased popularity in many countries in Europe, Asia and in the USA.

The method has been successfully employed in thousands of projects and the volume of improved soil from 1977 to 2010 exceeded 100 million cubic meters on the Japanese market alone. In the past three to four decades, traditional mechanical mixing has been improved to meet changing needs. Also new types of technologies have been introduced in the last 10 years and put into practice; e.g. high pressure injection and hybrid of mechanical and high pressure injection. The design procedures for various infrastructures were standardized by responsible organizations in Japan and revised several times.

The manuscript presents a State of the Art on the Deep Mixing methods in Japan that covers recent technologies, research activities and know-how in machinery, design, construction and quality control and assurance. The author hopes that the book will be a useful reference for academia and practitioners involved in deep mixing technology, regardless of local soil conditions and variation in applications.

11. REFERENCES

Architectural Institute of Japan (2006). Recommendations for Design of Ground Improvement for Building Foundations. Architectural Institute of Japan. (in Japanese).

Bruce, D.A., Bruce, M.E. and DiMillio, A.F. (2000) "Deep mixing: QA/QC and verification methods", Proceedings of the 4th Int. Conference on Ground Improvement Geosystems. Pp 11-22.

Cement Deep Mixing Method Association (1999). Cement Deep Mixing Method (CDM), Design and Construction Manual. (in Japanese).

Coastal Development Institute of Technology (2002). The Deep Mixing Method-Principle, Design and Construction-, A.A.Balkema Publishers. 123p.

Fire and Disaster Management Agency (1995). Criteria for a Specific Operation of Outdoor Storage Tank using Deep Mixing Method, Fire and Disaster Management Agency. No.150, 2p. (in Japanese).

Hosoya, Y., Nasu, T., Hibi, Y., Ogino, T., Kohata, Y. & Makihara, Y. (1996) "Japanese Geotechnical Society Technical Committee Reports: An evaluation of the strength of soils improved by DMM", Proceedings of the Second Int. Conference on Ground Improvement Geosystems. Vol.2. pp.919-924.

Japanese Geotechnical Society (2009). Practice for Making and Curing Stabilized Soil Specimens without Compaction. JGS 0821-2009, Japanese Geotechnical Society. Vol.1. pp 426-434 (in Japanese).

Kawasaki, T., Niina, A., Saitoh, S., Suzuki, Y. and Honjyo, Y. (1981) "Deep mixing method using cement hardening agent", Proceedings of the 10th Int. Conference on Soil Mechanics and Foundation Engineering. Vol.3. pp 721-724.

Kitazume, M. (2012). "Soil mixing performance in the 2011 Tohoku Earthquake," The Magazine of the Deep Foundations, Deep Foundations Institute. pp 77-78.

Kitazume, M. and Nagao, T. (2007) "Studies of reliability based design on deep mixing improved ground", Report of the Port and Airport Research Institute. Vol.46. No.1. pp 3-44 (in Japanese).

Kitazume, M. and Terashi, M. (2013). The Deep Mixing Method, CRC Press, 410 p.

Larsson, S. (2005) "State of Practice Report - Execution, monitoring and quality control -", Proceedings of the Int. Conference on Deep Mixing - Best Practice and Recent Advances. Vol.2. pp.732-785.

Ministry of Construction (1999) "Design and construction manual of liquefaction prevention techniques (draft)", Joint Research Report. 186p. (in Japanese).

Ministry of Land, Infrastructure, Transport and Tourism (2007). Technical Standards for Port and Harbour Facilities, (in Japanese).

Niina, A., Saitoh, S., Babasaki, R., Tsutsumi, I. and Kawasaki, T. (1977) "Study on DMM using cement hardening agent (Part 1)", Proceedings of the 12th Annual Conference of the Japanese Society of Soil Mechanics and Foundation Engineering. pp 1325-1328 (in Japanese).

Okumura, T. and Terashi, M. (1975) "Deep-lime-mixing method of stabilization for marine clays", Proceedings of the 5th Asian Regional Conference on Soil Mechanics and Foundation Engineering. Vol.1. No.1. pp 69-75.

Public Works Research Center (2004). Technical Manual on Deep Mixing Method for On Land Works, 334p. (in Japanese).

Saitoh, S., Suzuki, Y. and Shirai, K. (1985) "Hardening of soil improved by the deep mixing method", Proceedings of the 11th Int. Conference on Soil Mechanics and Foundation Engineering. Vol.3. pp 1745-1748.

Terashi, M. (2003) "The state of practice in deep mixing method. Grouting and ground treatment", Proceedings of the 3rd Int. Conference, ASCE Geotechnical Special Publication. No. 120. No.1. pp 25-49.

Terashi, M. (1983) "Problems and research orientation of the deep mixing method", Monthly Journal of Japanese Society of Soil Mechanics and Foundation Engineering, Tsuchi To Kiso. Vol.31. No.8. pp 75-83 (in Japanese).

Terashi, M. and Tanaka, H. (1981a) "Ground improved by deep mixing method", Proceedings of the 10th Int. Conference on Soil Mechanics and Foundation Engineering. Vol.3. pp 777-780.

Terashi, M. and Tanaka, H. (1981b) "On the permeability of cement and lime treated soils", Proceedings of the 10th Int. Conference on Soil Mechanics and Foundation Engineering. Vol.4. pp 947-948.

Terashi, M. and Tanaka, H. (1983) "Settlement analysis for deep mixing method", Proceedings of the 8th European Regional Conference on Soil Mechanics and Foundation Engineering. Vol.2. pp 955-960.

Terashi, M., Kitazume, M. and Yajima, M. (1985) "Interaction of soil and buried rigid structure", Proceedings of the 11th Int. Conference on Soil Mechanics and Foundation Engineering. Vol.3. pp 1757-1760.

Terashi, M., Okumura, T. and Mitsumoto, T. (1977) "Fundamental properties of lime-treated soils." Report of the Port and Harbour Research Institute. Vol.16. No.1. pp 3-28 (in Japanese).

Terashi, M., Tanaka, H. and Kitazume, M. (1983) "Extrusion failure of ground improved by the deep mixing method", Proceedings of the 7th Asian Regional Conference on Soil Mechanics and Foundation Engineering. Vol.1. pp v313-318.

Terashi, M., Tanaka, H. and Okumura, T. (1979) "Engineering properties of lime-treated marine soils and D.M.M. method", Proceedings of the 6th Asian Regional Conference on Soil Mechanics and Foundation Engineering. Vol.1. pp 191-194 .

Terashi, M., Tanaka, H., Mitsumoto, T., Honma, S. and Ohhashi, T. (1983) "Fundamental properties of lime and cement treated soils (3rd Report)", Report of the Port and Harbour Research Institute. Vol.22. No.1. pp 69-96 (in Japanese).

Terashi, M., Tanaka, H., Mitsumoto, T., Niidome, Y. and Honma, S. (1980) "Fundamental properties of lime and cement treated soils (2nd Report)", Report of the Port and Harbour Research Institute. Vol.19. No.1. pp 33-62 (in Japanese).

The Building Center of Japan (1997). Design and Quality Control Guideline of Improved Ground for Building, 473p. (in Japanese).

The Overseas Coastal Area Development Institute of Japan (2009). Technical Standards and Commentaries for Port and Harbour Facilities in Japan (English Version), 1028p.

The Ports and Harbours Association of Japan (2007). Technical Standards and Commentaries for Port and Harbour Facilities in Japan (Japanese Version), (in Japanese).

Tokimatsu, K., Mizuno, H. & Kakurai, M. (1996) "Building damage associated with geotechnical problems", Soils and Foundations. pp.219-234.

Uchida, A., Yamada, T., Odajima, N. and Yamashita, K. (2012) "Pile raft foundation with grid-form ground improvement subjected to the 2011 earthquake", Proceedings of 9th the Int. Conference on Urban Earthquake Engineering. CD.

Åhnberg, H. (2003) "Measured permeabilities in stabilised Swedish soils", Proceedings of the 3rd Int. Conference on Grouting and Ground Treatment. pp 622-633.