State-of-the-Practice in Ground Improvement in Singapore

J. Chu
Nanyang Technological University, Singapore

K. W. Leong
Keller Foundations (S.E. Asia) Pte Ltd, Singapore

ABSTRACT: In this State-of-the-practice report, the types of ground improvement methods that are commonly used in Singapore are reviewed. These include prefabricated vertical drains, preloading using fill or vacuum pressure with particular reference to land reclamation projects; granular sand densification using deep compaction or vibro compaction techniques; clayey soil improvement using dynamic replacement or vibro replacement methods such as stone columns and the others; jet grouting, deep cement mixing and other mixing methods. The construction process or related design issues for some of these methods are also discussed with case studies. Methods that have the potential to be used in Singapore for future ground improvement works are also presented.

1 INTRODUCTION

Ground improvement is one of the most commonly adopted geotechnical approaches in Singapore. The construction of many important infrastructures in Singapore such as airports, seaports, MRT lines, industrial parks, etc. would not have been possible without the ingenious use of ground improvement methods for these projects. The types of ground improvement methods adopted also cover a wide range. It would be a formidable task to give a comprehensive coverage of the state-of-practice to every ground improvement method adopted in Singapore. In this paper, we will only review the types of ground improvement methods that are most commonly adopted in Singapore. Some case histories in Singapore are also briefly presented. In general, the types of ground improvement methods can be classified as shown in Table 1, following the ground improvement classification system adopted by the ISSMGE Technical Committee TC211 (Chu et al. 2009c). Within this paper, the topics covered include prefabricated vertical drains, preloading using fill or vacuum pressure; granular soil densification using deep compaction or vibro compaction techniques, ground improvement using dynamic replacement or vibro replacement methods such as stone columns; jet grouting, deep cement mixing and other mixing methods. The selection of the topics is also affected by the experience of the reporters.

Table 1. Classification of ground improvement methods adopted by ISSMGE TC217 (after Chu et al. 2009c)

Category	Method	Principle
A. Ground improvement without admixtures in non-cohesive soils or fill materials	A1. Dynamic compaction	Densification of granular soil by dropping a heavy weight from air onto ground.
	A2. Vibrocompaction	Densification of granular soil using a vibratory probe inserted into ground.
	A3. Explosive compaction	Shock waves and vibrations are generated by blasting to cause granular soil ground to settle through liquefaction or compaction.
	A4. Electric pulse compaction	Densification of granular soil using the shock waves and energy generated by electric pulse under ultra-high voltage.
	A5. Surface compaction (including rapid impact compaction).	Compaction of fill or ground at the surface or shallow depth using a variety of compaction machines.
B. Ground improvement without admixtures in cohesive soils	B1. Replacement/displacement (including use of light-weight materials)	Remove bad soil by excavation or displacement and replace it by good soil or rocks. Some light-weight materials may be used as backfill to reduce load or pressure.
	B2. Preloading using fill (including the use of vertical drains)	Fill is applied and removed to pre-consolidate compressible soil so that its compressibility will be much reduced when future loads are applied.
	B3. Preloading using vacuum (including combined fill and vacuum)	Vacuum pressure of up to 90 kPa is used to pre-consolidate compressible soil so that its compressibility will be much reduced when future loads are applied.
	B4. Dynamic consolidation with enhanced drainage (including the use of vacuum)	Similar to dynamic compaction except vertical or horizontal drains (or together with vacuum) are used to dissipate pore pressures generated in soil during compaction.
	B5. Electro-osmosis or electro-kinetic consolidation	DC current causes water in soil or solutions to flow from anodes to cathodes which are installed in soil.
	B6. Thermal stabilisation using heating or freezing	Change the physical or mechanical properties of soil permanently or temporarily by heating or freezing the soil.
	B7. Hydro-blasting compaction	Collapsible soil (loess) is compacted by a combined wetting and deep explosion action along a borehole.
C. Ground improvement with admixtures or inclusions	C1. Vibro replacement or stone columns	Hole jetted into soft, fine-grained soil and back filled with densely compacted gravel or sand to form columns.
	C2. Dynamic replacement	Aggregates are driven into soil by high energy dynamic impact to form columns. The backfill can be either sand, gravel, stones or demolition debris.
	C3. Sand compaction piles	Sand is fed into ground through a casing pipe and compacted by either vibration, dynamic impact, or static excitation to form columns.

	C4. Geotextile confined columns	Sand is fed into a closed bottom geotextile lined cylindrical hole to form a column.
	C5. Rigid inclusions (or composite foundation)	Use of piles, rigid or semi-rigid bodies or columns which are either premade or formed in-situ to strengthen soft ground.
	C6. Geosynthetic reinforced column or pile supported embankment	Use of piles, rigid or semi-rigid columns/inclusions and geosynthetic girds to enhance the stability and reduce the settlement of embankments.
	C7. Microbial methods	Use of microbial materials to modify soil to increase its strength or reduce its permeability.
	C8 Other methods	Unconventional methods, such as formation of sand piles using blasting and the use of bamboo, timber and other natural products.
D. Ground improvement with grouting type admixtures	D1. Particulate grouting	Grout granular soil or cavities or fissures in soil or rock by injecting cement or other particulate grouts to either increase the strength or reduce the permeability of soil or ground.
	D2. Chemical grouting	Solutions of two or more chemicals react in soil pores to form a gel or a solid precipitate to either increase the strength or reduce the permeability of soil or ground.
	D3. Mixing methods (including premixing or deep mixing)	Treat the weak soil by mixing it with cement, lime, or other binders in-situ using a mixing machine or before placement
	D4. Jet grouting	High speed jets at depth erode the soil and inject grout to form columns or panels
	D5. Compaction grouting	Very stiff, mortar-like grout is injected into discrete soil zones and remains in a homogenous mass so as to densify loose soil or lift settled ground.
	D6. Compensation grouting	Medium to high viscosity particulate suspensions is injected into the ground between a subsurface excavation and a structure in order to negate or reduce settlement of the structure due to ongoing excavation.
E. Earth reinforcement	E1. Geosynthetics or mechanically stabilised earth (MSE)	Use of the tensile strength of various steel or geosynthetic materials to enhance the shear strength of soil and stability of roads, foundations, embankments, slopes, or retaining walls.
	E2. Ground anchors or soil nails	Use of the tensile strength of embedded nails or anchors to enhance the stability of slopes or retaining walls.
	E3. Biological methods using vegetation	Use of the roots of vegetation for stability of slopes.

A state-of-the-practice report would not be complete without a perspective of the emerging technologies that could influence the ground improvement practice in the future. A few ground improvement methods that have the potential to be adopted in Singapore in the near future are also presented.

A good ground improvement method should be based on sound concepts and working principles. The basic concepts are set by either engineers or specialist contractors based on their experience, knowledge of local geological conditions, available parameters, soil-structure interaction, criteria of strength and deformation, schedule and equipment availability. Very often, the basic concept of ground improvement is the combination of several techniques taking all the above criteria into account. Another important element in geotechnical design for ground improvement works is design parameters. Ground improvement is often carried out with very little knowledge of the ground. It is not uncommon in practice to obtain a specified end product in hundred thousands of cubic meters of soil based on the information provided by only a few kilograms of soil samples which are often disturbed. In addition to concepts and parameters, ground improvement also involves equipment and construction workmanship. A major part of the advances in ground improvement must be credited to the manufacturers of various ground improvement equipment. It is with the constant improvement in the equipment that we are able to push the boundaries of ground improvement technologies toward the direction of "better", "deeper", "faster", and "cheaper".

2 PRELOADING METHODS FOR CLAYEY SOIL

2.1 Introduction

It is well known that the compressibility and shear strength of soil can be greatly improved if the water content in the soil can be significantly reduced. One common method for improving soft soil is to reduce the water content of the soil through consolidation. For consolidation to occur there must be an increase in effective stress. This can be achieved by increasing the total stress or reducing the pore water pressure. The former is the so-called fill surcharge preloading method. The latter can be achieved through vacuum preloading. When a surcharge pressure is applied, the increase in the effective stress is dependent on the dissipation of excess pore water pressures generated as a response to the application of this surcharge. To accelerate the dissipation of pore water pressure, prefabricated vertical drains (PVDs) are normally used. PVDs are also used together with the vacuum preloading method to distribute vacuum pressure and facilitate the dissipation of pore water. Therefore, PVD techniques become part of the core technologies in the fill surcharge or vacuum preloading methods. PVDs have been used successfully in many ground improvement and land reclamation projects in Singapore and elsewhere (Hansbo 1981; Holtz et al. 1991; Akagi 1994; Bergado et al. 1993; Indraratna et al. 1994; Balasubraniam et al. 1995; Chu et al. 2000, 2006, 2009a, 2009b, 2009c; Choa et al. 1979, 2001; Bo et al. 2003, 2005; Arulrajah et al. 2009; Kitazume 2007; Varaksin and Yee 2007; Yan et al. 2009). The theories, design and construction methods for PVDs are the key technical issues in the preloading or consolidation methods for soft soil improvement.

Depending on how a preload is applied, the preloading methods can be subdivided into preloading using fill, preloading using vacuum pressure and combined fill and vacuum preloading methods, as described in Table 2. In addition to preloading, PVDs have also been used for some other relatively new methods such as dynamic consolidation for clays (Zheng et al. 2004; Chu et al. 2009c). In these cases, the main purpose of using PVDs is to reduce the drainage path so that the time taken for the dissipation of excess pore water pressure can be substantially reduced.

Table 2. Ground improvement methods using PVDs (modified from Chu et al. 2009c)

Method		Description / Mechanisms	Typical Applications	Advantages	Limitations
Preloading methods	A. Preloading using fill with vertical drains	A process to apply surcharge load prior to placement of structure or external loads to consolidate the soil until most of the primary settlement has occurred. PVDs are used to provide radial drainage and accelerate the rate of consolidation by reducing the drainage paths.	Applicable to soils having low permeability or when the compressible soil layer is thick.	Rate of consolidation can be greatly accelerated. The construction time can be controlled by adjusting the spacing of the drain.	The method may not be applicable when the construction schedule is very tight or when the ground is so soft that vertical drains cannot be installed.
	B. Vacuum preloading with vertical drains	Same as A, except the surcharge is applied using vacuum pressure. PVDs are used to distribute vacuum pressure and provide drainage. This method enables load of up to 90 kPa to be applied immediately without stability problem.	Same as A, except this method is particularly useful when there is a stability problem with fill surcharge. This method can also be used to extract polluted ground pore water, if required.	1). The method does not require fill material; 2). The construction period can be shorter, as no stage loading is required; 3). It may be more economical than using fill surcharge; 4) The vacuum brings immediate stability to the system.	1). This method causes inward lateral movement and cracks on the ground surface which may affect surrounding buildings or structures; 2). The vacuum pressure is limited to 50 - 90 kPa, depending on the system adopted.

C. Combined fill and vacuum preloading with vertical drains	A combination of A and B when a surcharge more than the limit of vacuum pressure (normally 80 kPa) is required.	The same as for A and B.	1). Construction time can be much reduced as compared to staged loading using fill surcharge alone; 2). The lateral movement of soil can be controlled by balancing the amount of vacuum and fill surcharge used. 3). The vacuum brings immediate stability to the system.	1). It is technically more demanding than A and B; 2). Data interpretation is also more complicated.

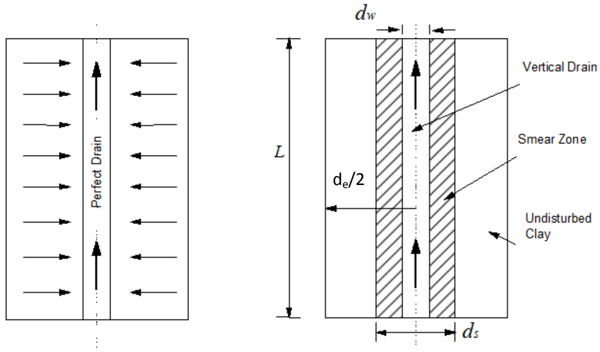
In the following, issues related to vertical drains with fill surcharge, vacuum preloading, and combined fill and vacuum surcharge methods for soft soil improvement are discussed.

2.2 Prefabricated vertical drains (PVDs)

A number of analytical solutions have been developed in the past for consolidation of ground improved with vertical drains (Carrillo 1942; Barron 1948; Yoshikuni and Nakanode 1974; Hansbo 1981; and Zeng and Xie 1989). Most of the theories adopted a "unit cell" model as shown in Fig. 1. Here, the band shaped drain is idealized into a circular drain with an equivalent diameter of $d_w = 2(a+b)/\pi$ as proposed by Hansbo (1979).

Radial consolidation theories such as those proposed by Carillo (1942) formed the basic equations for the analysis of radial consolidation of soil. When PVDs are used, other factors need to be taken into consideration. Two of the major factors are the smear effect and well resistance. When PVDs are installed in the soil, the penetration of the steel mandrel disturbs the soil surrounding the PVD. This smear effect causes a reduction in the permeability and coefficient of consolidation of the soil within the smear zone. When the discharge capacity of PVDs is limited, head loss will occur when water flows along the drain and delay the consolidation process. This unfavorable effect has been called the well resistance. Taking the smear effect and well resistance into account, the well-known Barron (1948) and Hansbo (1981) equations have been proposed and used for PVD design.

As an example to illustrate the parameters that affect the consolidation of soil using PVDs, Hansbo's equation (1981) is written as follows



(a) (b)

Figure 1 Unit cell model of (a) a perfect drain and (b) a drain with smear zone

$$U_h = 1 - \exp\left[\frac{-8T_h}{F(n)}\right] \tag{1}$$

$$F(n) \approx \ln(n) - 0.75 + \ln(s)\left(\frac{k_h}{k_s} - 1\right) + \pi z(2l - z)\frac{k_h}{q_w} \tag{2}$$

$$T_h = \frac{c_h t}{d_e^2} \qquad n = \frac{d_e}{d_w} \qquad s = \frac{d_s}{d_w} \tag{3}$$

where c_h is the coefficient of consolidation of soil in the horizontal direction, t is time, d_e is the diameter of soil cylinder dewatered by a drain which is related to the drain spacing: d_e =1.128s for a square grid and d = 1.05s for a triangle grid, F(n) is a function of d_e, d_w, the diameter of the smear zone, d_s, the horizontal permeability of the soil, k_h, the permeability of the smeared zone, k_s, the discharge capacity of the drain, q_w, the length of the drain, l, and the depth z. The last term in Eq. (2) represents the well resistance. It can be seen from Eqs. (1) and (2) that the factors affecting the consolidation of soil around PVDs are the soil parameters, c_h and k_h, the properties of the smear zone, d_s and k_s, and the properties of PVD, q_w. The effects of those factors will be discussed separately in the next section.

The design of PVDs looks simple. However, the performance of PVDs and the rate of consolidation predictions are affected by many factors. The main factors affecting the consolidation of soil around PVDs are the soil parameters, c_h and k_h, the properties of the smear zone, d_s and k_s, and the properties of PVD, q_w.

2.2.1 Soil parameters c_h and k_h

The determination of soil parameters is still one of the most challenging tasks facing geotechnical engineers. On one hand, we need to obtain a value for each soil parameter. On the other hand, few soil parameters are constant. For example, the coefficient of consolidation, c_v or c_h, is assumed to be a constant in either Terzaghi's or Barron's consolidation theory. However, in practice, neither c_v nor c_h for soft soil is a constant. Its value is affected by many factors, such as the overconsolidation ratio, the stress state, the fabric of the soil, and even the method of determination (Holtz and Kovacs 1981; Chu et al. 2002). As such, the selection of c_v or c_h has to be based on its in-situ stress conditions and the anticipated stress changes. Therefore, it is also necessary to establish relationships between the coefficient of permeability and void ratio, and relationships between the coefficient of consolidation and the stress state. A proper site investigation should be planned not only to determine the soil parameters but also to understand how the soil parameters vary with stress and loading conditions. The coefficient of permeability is another key parameter required for vertical drain design. However, it happens that the coefficient of permeability of soil is one of the most difficult soil parameters to be determined. This is partially because the coefficient of permeability of the soil has the widest range of variation among all the soil parameters. Its value can vary from 10^{-11} m/s for soft clay to 10^{-3} m/s for sand and gravel, a change of 10^{8} times. Although the permeability of the soil that has to be treated with vertical drains is normally low, the error involved in the permeability estimation can still range from 10 to 100 times. This is not unusual as the permeability of the same soil can change 10 to 100 times during the process of consolidation. An error of one order of magnitude in permeability can result in an error of the same order of magnitude in the time taken to achieve a specific degree of consolidation based on Terzaghi's consolidation theory as shown by Bo et al. (2003). Therefore, it makes sense economically to conduct some proper site investigation work and determine the soil parameters as accurately as possible.

Generally the consolidation parameters of soil can be determined using laboratory tests, in-situ tests, back calculation from field measurements, or a combination of them. In laboratory tests, the stress states and drainage conditions can be defined precisely and the variation of soil parameter with stress and consolidation process can be evaluated. However, the results are usually affected by sample disturbance. It is also time consuming to conduct laboratory consolidation tests. In-situ tests are normally relatively quick to conduct and therefore are more useful than laboratory tests in identifying the soil profile and characterizing the soil behavior over a large extent. However, in in-situ tests, the stress and drainage conditions are generally not well defined. The data interpretation from physical measurements to soil parameters are sometimes based on arbitrary assumptions or correlations which are established for a specific type of soil only. Therefore, when in-situ tests are adopted, laboratory tests may still be required to verify the assumptions and check the correlation relationships. The back-calculation from field measurements can provide a good check on the selection of design parameters. However, the back calculated value is only a factored parameter. It reflects not only the soil property, but also other factors, such as the disturbance to the soil during construction.

The types of laboratory and in-situ tests that are suitable to the determination of consolidation properties are summarized in Table 3. The settlement prediction for projects using vertical drains is the same as those without the use of vertical drains. Those methods are covered in many textbooks (e.g., Holtz and Kovacs 1981). As far as land reclamation or other types of geotechnical problems where the extent of load is much greater than the thickness of the compressible layer are concerned, the settlement predicted using one-dimensional analysis and parameters determined by laboratory tests is reasonable although it is not always reliable. Ground settlement should always be monitored as part of the ground improvement works.

Table 3 Types of test for measurement of consolidation properties

Type of test	Name of test	Parameter determined	Remarks
Laboratory tests	Oedometer test	c_v, k_v (indirect measurement[i]), C_c, C_r, σ_p', and C_α[ii]	Need good quality 'undisturbed' samples
	Rowe cell test	c_h and k_h (direct[iii] or indirect measurement)	
	Other consolidometers	c_h and k_h (direct or indirect measurement)	
In-situ tests	Piezocone dissipation test (CPTU)	c_h and k_h (indirect measurement)	Based on pore water pressure dissipation
	Pressuremeter or self-boring pressuremeter (SBPM) test	c_h and k_h (indirect measurement)	Based on lateral pressure change or pore water pressure dissipation
	Flat dilatometer test (DMT)	c_h and k_h (indirect measurement)	Based on lateral stress change
	Field permeability test (e.g., BAT permemeter)	k_h (direct measurement)	Using a piezometer
Back-analysis	Based on pore water pressure measurements	c_h (factored value)	Using piezometers
	Based on settlement measurements	c_h (factored value)	Using settlement gauges

Notes:
i. In this case, k_v is calculated based on the value of c_v.
ii. When secondary consolidation is measured.
iii. k_h is measured directly as part of the consolidation test.

2.2.2 Smear zone

Consolidation of soil around PVDs is affected by smear effect. However, it is not an easy task to determine the diameter of smear zone, d_s, and the permeability in the smear zone, k_s, because the smear effect is affected by many factors including the type of mandrel used, the method used to penetrate the mandrel, and the type of soil. The smear effect is due to not only the disturbance to the soil, but also the compressibility of the soil. To reduce the smear effect, the cross-section of the mandrel should be as small as possible. On the other hand, mandrel is a slender tube and it has to have a certain stiffness to be structurally stable. The influence of different types of mandrel and anchor shoes has been evaluated by Bo et al. (2003) and Basu and Prezzi (2007). In terms of method used to penetrate the mandrel, static pushing is better than vibration. Soil type is probably one of the most important factors. The smear effect in sensitive or cemented soil can be much greater than that in recent deposited soil (for example, clay fill used for land reclamation). A number of studies on smear effect have been carried out in the past (Hansbo 1979; 1981; Bergado et al. 1991; Madhav et al. 1993; Indraratna and Redana 1998; Chai and Miura 1999; Hird and Moseley 2000, Bo et al. 2003; Basu and Prezzi 2007; Abuel-Naga and Bouazza 2009). A summary of different studies has been given in Chu et al (2014). It should be pointed out that most of the studies on smear effect were based on laboratory tests using reconstituted soil. The use of reconstituted or remolded soil samples tends to underestimate the smear effect as the effect of destruction of soil structure or fabric cannot be reflected. Therefore, the smear effect should also be assessed by field measurements which unfortunately were rare.

One field study was carried out by Bo et al. (2003) at a reclaimed site in Singapore. Before the installation of PVD, piezometers were installed in marine clay in the vicinity of the drain. The pore pressures measured by the piezometers PP-240 and PP-239 installed at the same elevation, but 1.27 and 2.85 m radial distances, respectively, from the drain are shown in Fig. 8. The time in Fig. 2 was taken from the beginning of PVD installation. The change in the excess pore water pressure was a sign of disturbance due to smear. It can be seen that a large amount of pore pressure was measured by PP-240 which was 1.27 m away from the drain. The drain used had a width of 100 mm and a thickness of 4 mm. Thus the equivalent diameter d_w is 66 mm. However, a point 1.27 m away from the drain is $32d_w$. One may argue that this is an indication of the transition zone, not the smear zone. If this is the case, the boundary of the transition zone can be as far as 2.85 m away from the drain. This is because there was still pore water pressure generation at this distance as shown in Fig. 2.

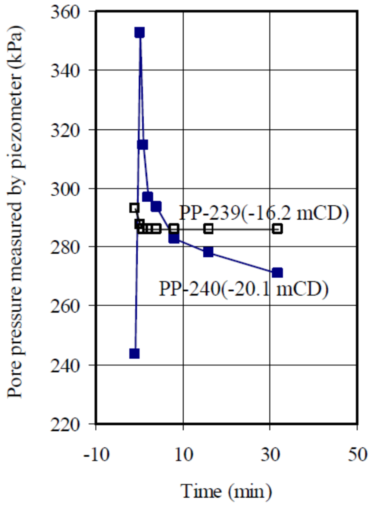


Figure 2. Change in pore water pressure due to penetration of mandrel (after Bo et al., 2003)

In the field tests reported by Bo et al. (2003), the change in the field permeability of soil was also measured in-situ using BAT permeameter with a tip of 30 mm in diameter and 40 mm in length. The tests were carried out at a radial distance of 0.3

m and 0.5 m and 1.0 m away from the PVD. It was concluded from the study that reduction in permeability ranges from 1.8 to 11.0 times. Hence for intact soil in-situ, the real reduction in permeability can be much greater than those suggested by the laboratory studies for smear effect.

For Singapore marine clay, the diameter of a smear zone, d_s, may be taken as 2 to 6 times of the equivalent dimeter of the mandrel, d_m, as a rule of thumb. The reduction in the permeability can be taken as 2 to 4 times depending on the soil types and OCR of the soil (Chu and Raju 2014). The dimension of a typical rectangular mandrel is 120 mm long by 60 mm wide (Bo et al. 2003) which gives an equivalent diameter of the mandrel, d_m, 115 mm. Hence the diameter of the smear zone, d_s, should be in the range of 230 to 690 mm. For a drain spacing of 1 m, it means more than half of the soil can be of disturbed. Therefore, it may not always be beneficial to use a close drain spacing to reduce the consolidation time unless the soil to be consolidated is deposited recently. For one project reported by Chu et al. (2002), the back calculated c_h based on field monitoring data was even smaller than the c_v determined by laboratory oedometer tests. This could be because of smear effect as discussed by Chu et al. (2002).

2.2.3 Well resistance

Well resistance refers to the finite permeability of the vertical drain with respect to the soil. Head loss occurs when water flows along the drain and delays radial consolidation. Theoretically, the well effect is modeled by the last term of Eq. (2): $\pi z(2l-z)k_h/q_w$. Therefore, the well resistance is controlled by the length of the drain, the discharge capacity of the drain q_w, and the permeability of the soil k_h. However, if q_w is sufficiently large, then this term $\pi z(2l-z)k_h/q_w$ can be small enough to be ignored. The good news is there are PVD products that can provide enough q_w to make the well effect insignificant. The required value of q_w for well resistance to be ignored will be discussed in the next section.

It should be pointed out that the discharge capacity can be significantly reduced when PVDs experience kinking under large deformation and thus lead to well resistance. In this respect, circular PVDs have shown to be more favorable in comparison to band or wick type PVDs, especially when vacuum pressure is employed.

2.2.4 Selection of PVDs

The quality and suitability of the drains play a key role in the whole ground improvement scheme involving PVDs. Different design situations require different types of PVDs. For example, it is not necessary to use a vertical drain with a high discharge capacity value if the drain is short. The drain filter should also match the soil type. The unit price of vertical drain is another important consideration besides meeting the design requirements. A considerable saving can be achieved without sacrificing the performance of the drain, if the control factors for a vertical drain can be identified and the design requirements are specified accordingly. The other factors that control the selection of vertical drain, apart from the cost, include discharge capacity, compatibility of the filter with the soil to be improved, and the tensile strength of drain.

As discussed above, the well resistance may be ignored if the discharge capacity of PVD, q_w, is adequate. The discharge capacity of drain, q_w, required is affected by the permeability of soil, k_s, and the discharge length or the depth of PVD installation, l_m. Studies have shown that for well resistance to be ignored, the required discharge capacity, q_{req}, has to meet the following requirement (Chu et al. 2004):

$$q_{req} \geq 7.85 F_s k_h l_m^2 \tag{4}$$

where: q_{req} = the required discharge capacity, F_s = factor of safety to take account the factors influencing the discharge capacity such as kinking. Normally $F_s = 4 \sim 6$ is suggested.

Inequality (4) reflects the fact that the larger the k_h or the longer of the drain, the larger the discharge capacity is required.

In addition to having sufficient discharge capacity, the filter of PVDs should have adequate apparent opening size (AOS). On one hand, the AOS has to be small enough to prevent the fine particles of the soil from entering the filter and the drain. On the other hand, the AOS cannot be too small as the filter has to provide sufficient permeability. A commonly used criterion is given by Carroll (1983):

$$O_{95} \leq (2 \sim 3)\, D_{85} \quad \text{and} \quad O_{50} \leq (10 \text{ to } 12)\, D_{50} \tag{5}$$

where O_{95} is the AOS of filter, O_{50} is the size which is larger than 50% of the fabric pores, D85 and D_{50} refer to the sizes for 85% and 50% of passing of soil particle by weight. $O_{95} \leq 0.075$ mm, or 75 μm, is often specified for prefabricated vertical drains. A more relaxed criterion:

$$O_{95} \leq (4 \sim 7.5)\, D_{85} \tag{6}$$

may be applicable to soft clay in Singapore (Chu et a. 2004) and Bangkok (Bergado et al. 1993). Past experience also indicates that the mass to area ratio should be generally larger than 90 g/m^2 (Bo et al. 2003).

PVDs should have adequate tensile strength so that it can sustain the tensile load applied to it during installation. Therefore the strength of the core, the strength of the filter, the strength of the entire drain, and the strength of the joint need to be specified, normally at both wet and dry conditions. According to Kremer et al. (1983), a drain must be able to withstand at least 0.5 kN of tensile force along the longitudinal direction without exceeding 10% in elongation. It is quite common nowadays to specify the tensile strength of the whole drain at both dry and wet conditions to be larger than 1 kN at a tensile strain of 10%.

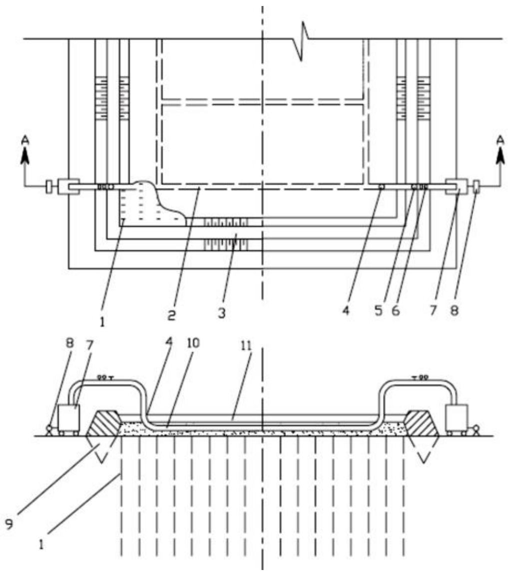
Some examples of design specifications and testing methods to measure discharge capacity, AOS and tensile strength of PVDs have been discussed in Bo et al. (2003). It is worthwhile to mention that a few design codes have also been put in use in different countries. Examples are the European Standard on Execution of Special Geotechnical Works — Vertical Drainage (BS EN 15237, 2007), Australian Standards AS8700: Execution of Prefabricated Vertical Drains (2011), and the Chinese design codes JTJ/T256-96 (1996) to control the practice for installation of PVDs and JTJ/T257-96 (1996) to set the quality inspection standard for PVDs. The use of these codes and standards are important in maintaining the quality standards of ground improvement works. In the past, a number of ASTM D-series standards were often used as the standards for vertical drain testing. However, some of those ASTM standards are not specifically written for vertical drains. Therefore, the testing procedures stipulated in these standards may not be the most suitable methods.

2.3 Vacuum preloading

2.3.1 Vacuum consolidation systems

When the ground is very soft or when the fill surcharge has to be applied in stages to maintain the stability of the fill embankment, the vacuum preloading method becomes a good alternative. Vacuum preloading is also used when there is no fill or the use of fill is costly, when there is no space on site to place the fill and when slurry or soft soil is used as fill for reclamation. The idea of vacuum preloading was proposed by Kjellman in 1952. Since then, the vacuum preloading method has evolved into a mature and efficient technique for the treatment of soft clay. This method has been successfully used for many ground improvement or land reclamation projects in Singapore as well as all over the world (Holtz 1975; Chen and Bao 1983; Cognon 1991; Bergado et al. 1998; Chu et al. 2000; Chu and Yan, 2005b, Yan and Chu, 2003, 2005; Indraratna et al. 2011; Chu et al. 2016).

The schematic arrangement of the vacuum preloading system adopted in Singapore and China is shown in Fig. 3. PVDs are normally used to distribute vacuum load and discharge pore water. The ground improvement work using the vacuum preloading method is normally carried out as follows. A 0.3 m sand blanket is first placed on the ground surface. PVDs are then installed on a square grid at a spacing of 1.0 m in the soft clay layer. Corrugated flexible pipes (50 to 100 mm diameter) are laid horizontally in the sand blanket to link the PVDs to the main vacuum pressure line. The pipes are perforated and wrapped with a nonwoven geotextile to act as a filter layer. Three layers of thin PVC membranes are laid to seal each section. Vacuum pressure is then applied using jet pumps. The size of each section is usually controlled in the range of 5,000 to 10,000 m^2. Field instrumentation is an important part of the vacuum preloading technique, as the effectiveness of vacuum preloading can only be evaluated using fielding monitoring data. Normally piezometers, settlement gauges and inclinometers are used to measure the pore water pressure changes, the settlement at ground surface and/or different depths in the soil and the lateral displacement. More details are presented in Chu et al. (2000) and Yan and Chu (2003).



1, drains; 2, filter piping; 3, revetment; 4, water outlet; 5, valve; 6, vacuum gauge; 7, jet pump; 8, centrifugal pump; 9, trench;10, horizontal piping; 11,

Figure 3. Vacuum preloading system used in China (after Chu et al. 2000)

In Europe, the Menard Vacuum Consolidation system has been developed in France by Cognon (1991). The details of this system can be found in Varaksin and Yee (2007). The general principle following this method is presented in Fig. 4. The uniqueness of this system is the dewatering below the membrane which permanently keeps a gas phase between the membrane and the lowered water level. Therefore, the Menard Vacuum Consolidation system adopts a combined dewatering and vacuum preloading methods to maintain an unsaturated pervious layer below the membrane. The vacuum preloading method may not work well when the subsoil is inter-bedded with sand lenses or permeable layers that extend beyond the boundary of the area to be improved, such as the improvement of soft soil below sand fill for reclaimed land. In this case, a cut-off wall is required to be installed around the boundary of the entire area to be treated. One example is given by Tang and Shang (2000), in which a 120 cm wide and 4.5 m deep clay slurry wall was used as a cut-off wall in order to improve the soft clay below a silty sand layer. However, installation of cut-off walls is expensive when the total area to be treated is large. One solution to this problem is to connect the vacuum channel directly to each individual drain. This so-called BeauDrain system has been developed in the Netherlands (Kolff et al. 2004). This method has evolved in the past few years and the one of the later version is shown in Fig. 5. In this method, the top of each vertical drain is connected to a plastic pipe as shown in Fig. 5a & 5b. In this way, the channel from the top of the PVD to the vacuum line is sealed using the plastic pipe and thus go through a sand layer without causing leak in vacuum. A special connector as shown in Fig. 5b is used for this purpose. The plastic pipes are connected directly with the vacuum line at the ground surface as shown in Fig. 5c. Thus, a sand blanket and membranes as used in the conventional vacuum methods as shown in Figs. 3 and 4 are not required. This method has been used for the construction of the new Bangkok Suvarnabhum International Airport (Seah 2006; Saowapakpiboon et al. 2008) and other projects (Chai et al. 2008). One shortcoming of this method is that it is difficult to achieve a high vacuum pressure in soil. This could be caused by two factors. The first is the difficulty to ensure every drain is completely sealed. The second is the possible head loss in the sealed plastic pipe (see Fig. 5a). This method also requires a more detailed soil profile as the length of each PVD has to be predetermined to match the depth of the clay layer at each PVD location. The production rate is also thus lower.

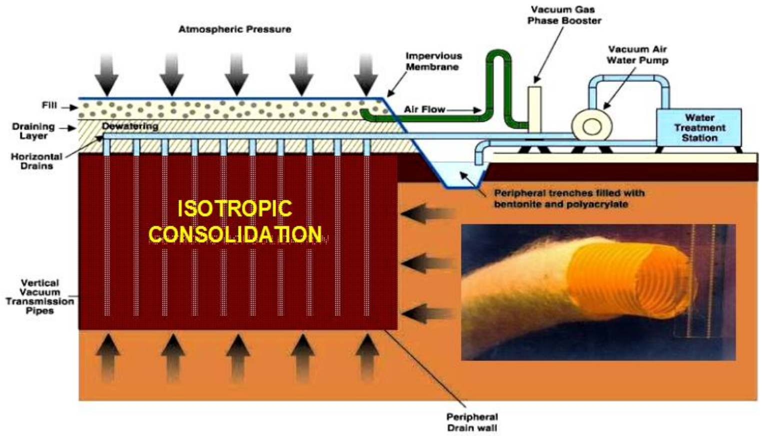


Figure 4. The Menard vacuum consolidation system (After Varaksin and Yee 2007)

(a) (b) (c)

Figure 5 BeauDrain vacuum preloading system (a) Concept (Courtesy of Cofra, Holland); (b) Direct connection of PVD with plastic pipe for vacuum application; and (c) Connection of plastic pipes to a vacuum pump (after Chu et al. 2015)

Another method to do away with the membrane is to use the so-called low level vacuum preloading method (Yan and Cao 2005). This method is schematically illustrated in Fig. 6 (Chu et al. 2008). When clay slurry is used as fill for land reclamation, the vacuum pipes can be installed at the seabed or a level a few meters below the ground surface. In this way, clay slurry fill can be placed on top of the vacuum pipes. As clay has a low permeability, the fill material will provide a good sealing cap and membranes will not be required. However, this method is not problem-free. Tension cracks can develop on the surface when the top layer is dried. The vacuum pressure may not be distributed properly unless a drainage blanket is used at the level where the drainage pipes are installed or the individual drains are connected to the vacuum pipes directly. It is also difficult to install drainage pipes or panels underwater. Nevertheless, this method does not require the construction of inner dikes for subdivision and thus cuts down the project costs and duration substantially.

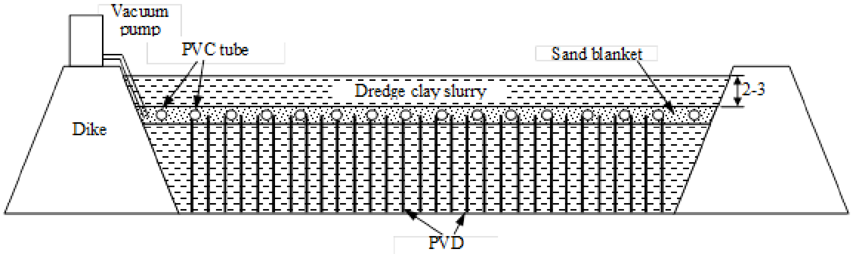


Figure 6 Membraneless vacuum preloading method (After Chu et al. 2008)

2.4 Combined vacuum and fill surcharge consolidation

One limitation of the vacuum preloading method is that the nominal vacuum pressure can only be 80 kPa. When higher surcharge is required, a combined vacuum and fill surcharge method can be adopted in which a fill surcharge can be applied after the soil has gained adequate strength under the vacuum load. One example is given in Fig. 7 where 3.5 m of fill was placed after 80 kPa of vacuum pressure was applied for 1.5 months. The ground settlement versus time curve is also shown in Fig. 7. For a detail description of the project, see Yan and Chu (2005). In this project, the pore water pressures at different depths were also measured and the pore water pressure distribution profiles are shown in Fig. 8. It should be noted that with respect to the initial pore water pressure profile, there was an almost uniform reduction in the pore water pressure over the entire depth of 16 m at the end of consolidation. It is an indication that the well resistance was insignificant in this case. Within the first 30 days, there was a pore water pressure reduction of about 20 and 47 kPa at 3 and 16.5 m deep respectively. When the fill surcharge of about 60 kPa was applied at 45 days, the pore water pressure should have increased by 60 kPa. The pore water pressures measured at 60 days indicate a reduction of pore water pressure of 70 and 65 kPa at 3 and 16.5 m deep respectively. This is more than the pore water pressure reduction in the first 30days! It implies that the consolidation under a combined load is more efficient than that under vacuum load alone. This could be partially attributed to the increase in hydraulic gradient after a positive increase in pore water pressure.

The combined vacuum and fill surcharge method offers several other advantages over either the vacuum or fill surcharge method alone. Firstly, it cuts down the construction time as the vacuum pressure can be applied relatively quickly and the subsequent fill surcharge can be applied much quicker too as the soil has been adequately consolidated under the vacuum load. Secondly, it provides a way to control the lateral displacement. Under vacuum pressure, an inward lateral displacement is created. On the other hand, under fill surcharge, the soil will move laterally outward. Lateral displacements are undesirable most of the time. If a loading program is designed properly, the vacuum and fill surcharge can be applied in a way to control the lateral displacement within a certain limit (Yan and Chu 2005). For this reason, this method is particularly suitable to be used when preloading has to be carried out near a retaining wall, an embankment or a dike.

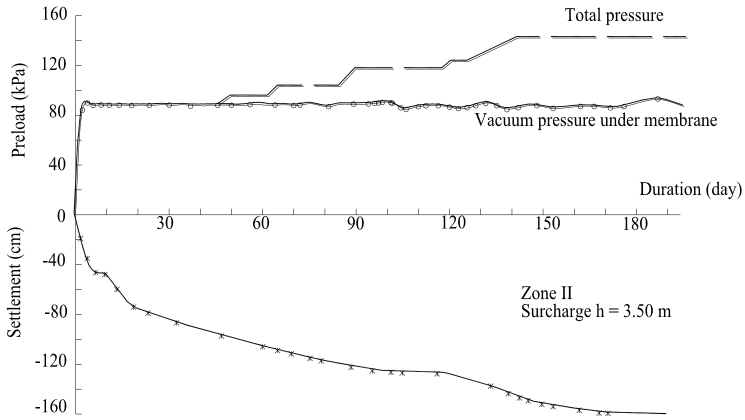


Figure 7 Loading sequence and ground settlement measured at Section II (after Yan and Chu 2005)

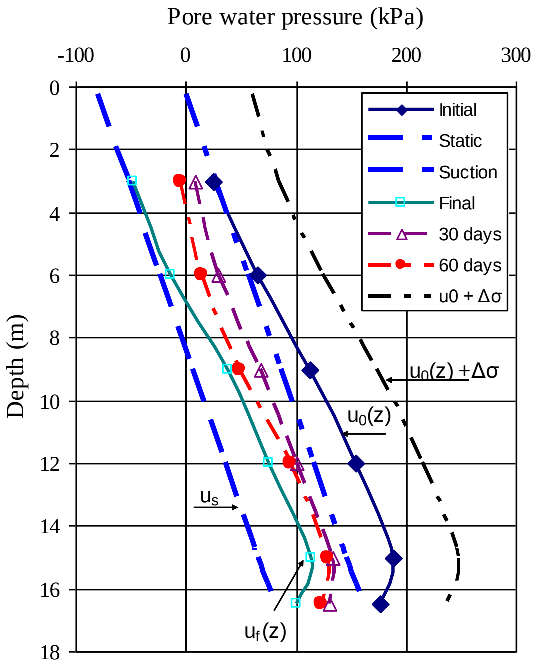


Figure 8 Pore water pressure distributions with depth at Section II (after Yan and Chu 2005)

2.5 *Performance evaluation*

A ground improvement project using preloading is usually carried out until the required degree of consolidation is obtained. Assessment of the degree of consolidation of the soil therefore becomes one of the most important tasks for construction control. One of the most commonly adopted methods for assessing the degree of consolidation of soil is by means of field instrumentation using settlement or pore water pressure data. For this reason, field instrumentation is normally required to monitor settlements and pore water pressures at different elevations as well as ground water tables, lateral displacement, and earth pressure etc. Some of the commonly used instruments are described in Bo et al (2003). Case studies and other instrumentation issues are discussed by many researchers (e.g., Bo et al. 2003, Arulrajah et al. 2009). For projects using PVDs, in particular those use vacuum preloading, it is important to measure the pore water pressures at several depths in the soil to obtain a pore water distribution profile as shown in Fig. 8 as this is the most effective way to visualize whether consolidation is progressing.

The degree of consolidation is normally calculated as the ratio of the current settlement to the ultimate settlement. However, for a ground improvement project, the ultimate settlement is unknown and has to be predicted. Although consolidation settlement can be estimated based on laboratory oedometer tests, the prediction by this method is normally not very reliable. Methods to estimate the ultimate settlement based on field settlement monitoring data are also proposed. Among them, the Asaoka's (1978) and hyperbolic (Sridharan and Rao 1981; Tan et al. 1991, Tan, 1993) methods are commonly used. Once the pore water pressures at different depths are measured during preloading, the initial and final pore water pressure distributions with depth can be plotted as shown in Fig. 8 as an example. The typical pore water pressure distribution profiles for a combined vacuum and fill surcharge preloading case are shown schematically in Fig. 9. Using Fig. 9, the average degree of consolidation, U_{avg}, can be calculated as:

$$U_{avg} = 1 - \frac{\int [u_t(z) - u_s(z)]dz}{\int [u_0(z) - u_s(z)]dz} \tag{7}$$

and

$$u_s(z) = \gamma_w z - \sigma, \quad \text{kPa}$$

In Eq. 7, $u_0(z)$ = the initial pore water pressure at depth z; $u_t(z)$ = the pore water pressure at depth z at time t; $u_s(z)$ is the suction line, z = depth, γ_w = unit weight of water, and s = suction applied. The value of s is normally assumed to be 80 kPa. The integral in the numerator in Eq. 7 is the area between the curve $u_t(z)$ and the suction line $u_s(z)$, and the integral in the denominator the area between the curve $u_0(z)$ and the suction line $u_s(z)$.

As an example, the settlement and pore water pressure data presented in Figs. 7 and 8 are used to estimate the degree of consolidation at the end of the preloading. Asaoka's method was applied to predict the ultimate settlements, S_∞, using the ground settlement data shown in Fig. 8. The results are given in Table 4. Using the pore water pressure distribution profile shown in Fig. 8 and Eq. (7), the average degree of consolidation was estimated and the value is given in Table 4. The degree of consolidation measured using the pore water pressure data is smaller than those by displacements. This is typical for the reasons explained by Chu and Yan (2005a).

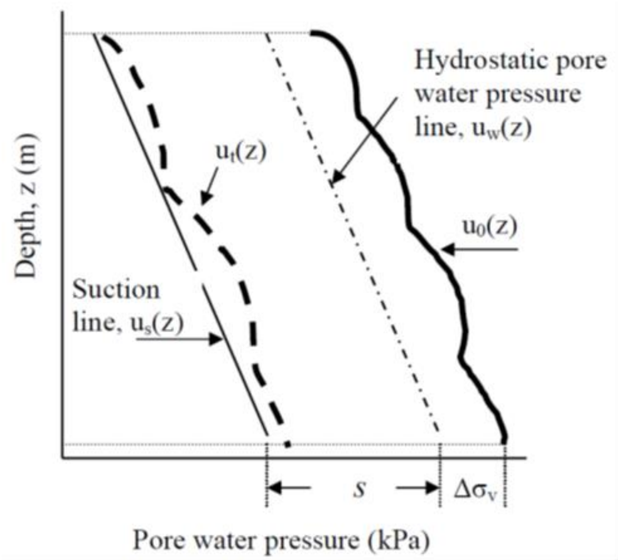


Figure 9 Schematic illustration of pore water pressure distributions versus depth under combined surcharge and vacuum load (After Chu and Yan 2005a)

Table 4 Ultimate settlement and Degree of consolidation estimated by different methods

Section	Asaoka's method		Based on Pore pressure
	S_∞ (m)	U_f (%)	U_f (%)
II	1.84	87	82

One concern of the method depicted in Fig. 9 is that the random uncertainties in the pore water pressure measurements as the distance between the pore pressure transducers and the PVDs can affect the pore pressure distribution profile. This is true only when the depth of PVD is relatively short, say less than 5 m. This is because when a random variable varies over a long distance, the overall effect of the random variation over the entire distance reduced greatly due to a statistical property called spatial variance reduction (Vanmarcke 1977). This explains why the method illustrated in Eq. (7) has worked well for a number of projects (Chu et al. 2000; Yan and Chu 2003; 2005; Chu et al. 2009a).

2.6 Case histories

2.6.1 Changi East land reclamation projects

The Changi East reclamation project was carried out in five phases along the foreshore of the east coast of Singapore. The water depths in the reclaimed area ranged from 5 to 15 m. The project involved hydraulic placement of 272 million m3 of sand onto soft seabed marine clay up to 50 m thick. A linear total of 170,000 km of prefabricated vertical drains (PVDs) were installed for accelerating the consolidation process of the underlying soft marine clay. The ground improvement works covered a total area of approximately 1200 ha. Pilot tests with full-scale field instrumentations as well as laboratory and in situ tests were carried out to verify the design, check the effectiveness of the ground improvement works using PVDs, and establish the most suitable drain spacing. Field monitoring data obtained from both the pilot tests and the reclamation works are presented and interpreted. Degree of consolidation was calculated based on both settlement and pore pressure data. The site conditions of one pilot test are shown in Fig. 10 with soil properties given in Table 5. The monitoring data for Lot 1.5 with PVD spacing of 1.5 m in a square pattern are shown in Fig. 11.

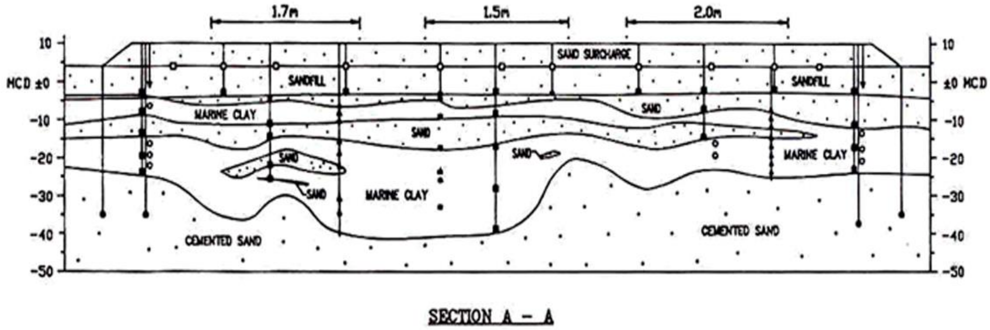


Figure 10 Cross-section of the pilot test area (after Chu et al. 2009a)

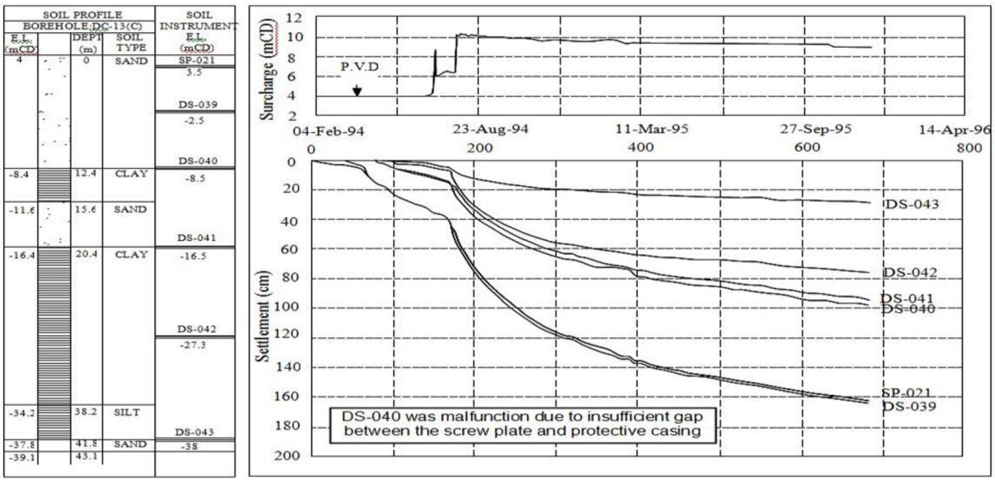


Figure 11 Settlement data for Lot 1.5 with PVDs installed at a 1.5 m spacing (Chu et al. 2009a)

Table 5. Range of physical and compressibility parameters of Singapore Marine Clay at Changi, Singapore (after Chu et al. 2009a)

Parameter	Upper marine clay	Intermediate layer	Lower marine clay
Unit weight, γ (kN/m^3)	14.2-15.7	18.6-19.6	15.7-16.7
Water content (%)	50-85	10-40	40-66

Liquid limit (%)	70-95	30-70	60-90
Plastic limit (%)	20-28	18-20	20-30
Initial void ratio, e_o	1.8-2.2	0.5-0.9	1.1-1.7
Specific gravity, G_s	2.60-2.72	2.68-2.76	2.70-2.75
Compression index, C_c	0.6-1.5	0.2-0.3	0.4-1.0
Recompression index, C_r	0.1-0.2	0.02-0.15	0.05-0.2
Secondary compression index, C_α	0.012-0.025	0.004-0.023	0.012-0.023
OCR	1.5-7.0	3.0-4.0	1.8-2.0
Vertical coefficient of consolidation at NC state, c_v (m^2/yr)	0.5-1.7	-	0.5-2.3
Horizontal coefficient of consolidation at NC state, c_h (m^2/yr)	2.0-4.0	-	3.0-6.0
Coefficient of permeability (m/s)	10^{-8}-10^{-9}	-	10^{-9}-10^{-10}

The pore water pressures monitored for the no PVD area, Lot X, and Lot 1.5 at different durations are shown in Fig. 12. To verify the piezometer readings, pore water pressures measured by CPT dissipation tests until full excess pore pressure dissipation are also shown in Fig. 12. It can be seen that good agreement between the two was achieved. The predictions of the ultimate settlement and degree of consolidation for the 4 Lots (including Lot X) in the pilot tests are summarized in Table 6.

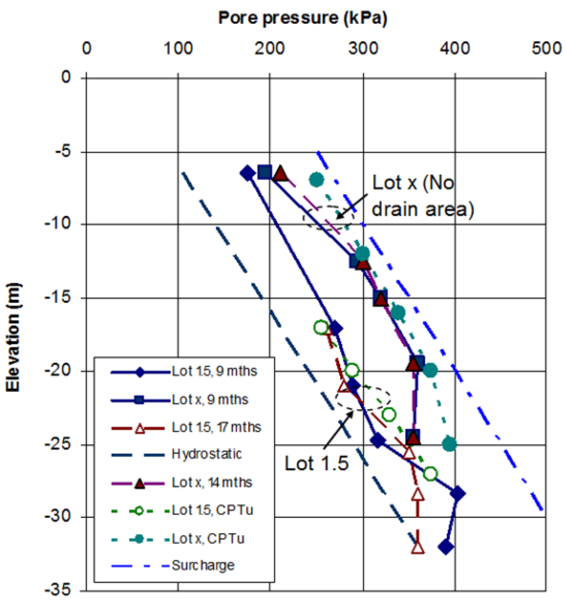


Figure 12. Pore water pressure profiles for Lots X and Lot 1.5 (after Chu et al. 2009a)

Table 6. Ultimate settlement and degree of consolidation estimated for the Pilot test

Pilot test section	Lot 1.5	Lot 1.7	Lot 2.0	Lot X
Total clay thickness (m)	21.0	16.75	11.4	22.0
Measured final settlement (cm)	163.5	176.0	59.5	77.0
Estimated ultimate settlement (cm)	180	190	73	100
Degree of consolidation based on settlement (%)	91	93	82	77
Degree of consolidation based on pore pressure (%)	87	-	-	-
Estimated residual settlement (cm)	16.5	14.0	13.5	23.0
Rate of settlement after 18 mth preloading (cm/mth)	1.7	1.3	0.7	1.5
Strain rate after 18 mth preloading (%/mth)	0.08	0.08	0.06	0.07
Rate of secondary compression (%/mth)	0.004	0.004	0.004	0.004

2.6.2 Ground improvement using membraneless vacuum preloading

A pilot study using a combined vacuum preloading and fill surcharge method for the ground improvement of a marine clay layer below the reclaimed sand fill is presented. Because of the presence of the sand fill, a membraneless vacuum method, the so-called Beaudrain-S, was applied to avoid the use of a cutoff wall which would be required in the conventional vacuum method were applied. For this method, each PVD has to be connected to a 16 mm sealed HDPE vacuum tube through a T-coupling as shown in Fig. 13. For this reason, the lengths of each PVD and the HDPE tube respectively has to be predetermined and connected before installation. Each PVDs with the T-coupling was then connected to the vacuum distribution pipes as shown in Fig. 13. PVD spacing of 1.0 m was used. Fill surcharge 4 m high was added several weeks after the vacuum load was applied. The project site had a non-uniform soft clay layer varying from 4 to 17 m in thickness. The ground settlement, settlements and pore pressure at different evaluations in the soil were measured during the preloading. The settlements at different elevations as measured at one point is shown in Fig. 14 and pore pressures data at another location are shown in Fig. 15. This field trial was not entirely successful as leakage of vacuum pressure was detected at several locations due to an incorrect estimation of the thicknesses of the sand fill at those locations.

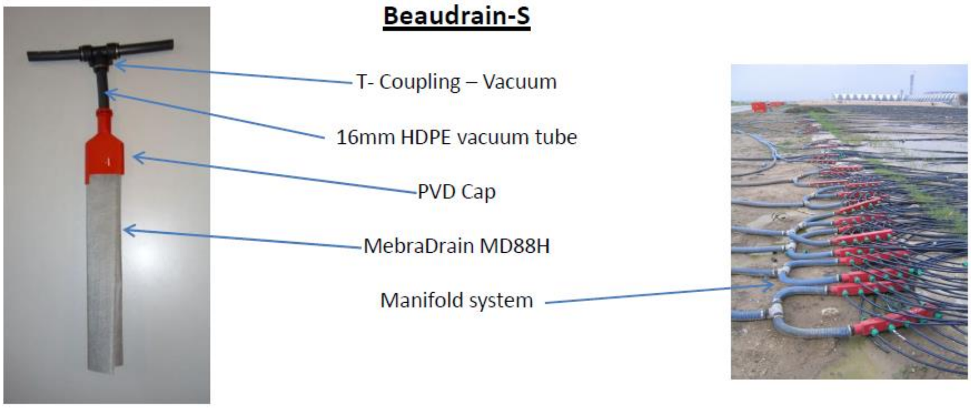


Figure 13 Beaudrain-S membraneless vacuum preloading system

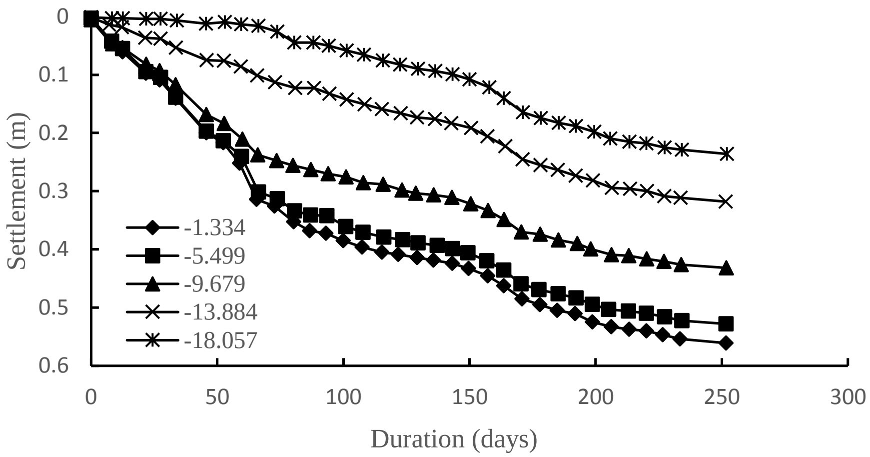


Figure 14 Settlement versus duration curves monitored at different elevations

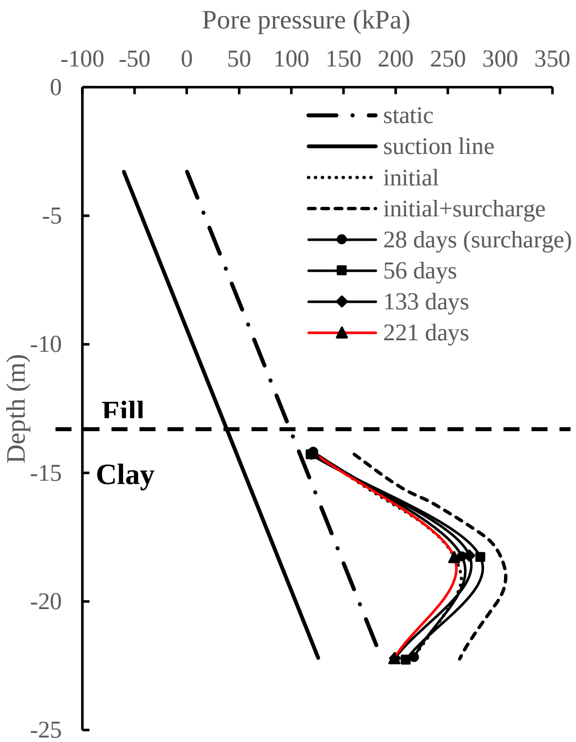


Figure 15 Distributions of pore water pressure versus depth at different durations

2.7 Summary

An overview on preloading methods using PVDs with either fill surcharge or vacuum preloading is presented. The main points discussed are summarized as follows:

(1) The design of PVDs is straight forward. Still reliable prediction of the performance of PVDs are difficult to achieve. This is because there are too many factors (or uncertainties) involved in the determination of the design parameters. To reduce the uncertainties, the designer should be willing to obtain the most reliable soil parameters, c_h and k_h. Both c_h and k_h are stress history or stress state dependent parameters and thus have to be selected based on the stress conditions.

(2) The design of PVDs is also affected by the smear effect. The smear zone properties are difficult to determine as it is affected by the mandrel used, the method used to penetrate the mandrel and the type of soil. As a rule of thumb for local marine clay, the diameter of the smear zone, d_s, may be taken as 2 to 6 times of the equivalent dimeter of the mandrel, d_m, and the reduction in the permeability can be taken as 2 to 4 times depending on the soil types and OCR of the soil.

(3) Well resistance effects may be ignored if the discharge capacity, q_w, is sufficiently large. The required q_w value may be calculated as $q_{req} \geq 7.85 F_s k_h l_m^2$, where F_s is a factor of safety to consider effect of buckling and large deformation of PVD on q_w. The selection of PVD should be based on q_w, tensile strength of PVD, and the AOS of the filter.

(4) The vacuum preloading system normally requires membrane to be used to seal the soil to be consolidated, such as the China and Menard systems. Membraneless vacuum systems have also been developed. This includes the BeauDrain system in which each PVD is connected directly to the vacuum pump through plastic pipes and the low level vacuum preloading method.

(5) The combined vacuum and fill surcharge method offers several advantages over either the vacuum or fill surcharge method alone: (a) it cuts down the construction time and (2) it provides an effective way to control the lateral displacement.

(6) To evaluate the degree of consolidation, both settlement and pore water pressure should be used. Pore water pressures at a few depths in the clay should be measured to plot the pore water pressure versus depth profile which enables the effectiveness of vacuum consolidation to be evaluated.

3 DENSIFICATION METHODS FOR GRAUNLAR SOIL

3.1 Background

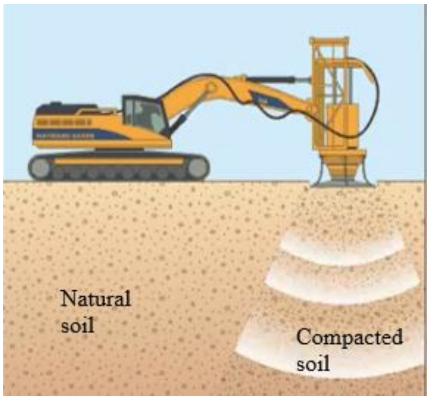
Densification of sand or gravel is another commonly used ground improvement methods in Singapore. When land is reclaimed using sand fill for infrastructure developments, the hydraulically placed sand fil is often in a loose state with relative densities in the range of 30 to 40% (Raju and Sondermann, 2005). Densification of the sand fill is often necessary to increase the relatively density so as to increase the shear resistance and reduce the compressibility of the sand fill.

3.2 Types of densification techniques

Based on the required depth of compaction, various densification techniques may be employed. Therefore, one possible way to classify the densification techniques is, as shallow and deep compaction techniques. Shallow compaction includes surface compaction with rollers, high energy impact compaction, and rapid impact compaction (RIC). Deep compaction includes dynamic compaction (DC) and vibro compaction (or vibro floatation). Surface compaction is only applicable to densification of soil in the shallow layer, usually from 1 to 2 m using conventional compaction machines to up to 4 m using the rapid impact compaction (Chu et al. 2009). DC is normally used for the compaction of sand fill less than 10 m. On the other hand, vibro compaction is suitable for the densification of relatively deeper sand deposits.

3.2.1 Shallow compaction

For surface compaction, equipment such as tampers, smooth-wheeled rollers, pneumatic-tyred rollers, impact rollers and vibratory rollers can be used for granular fills. RIC is relatively new technique that can be adopted for improvement depth of less than 5 m. A RIC machine as used for a land reclamation project in Jurong Island in Singapore is shown in Figure 16. It can be used in narrow areas with height constraints. However, it is not suitable if the structures are present within 6m of operation and cannot be used under water (Narendranathan and Lee, 2015).

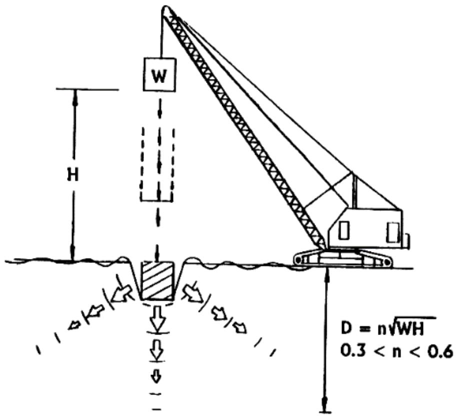


(a) (b)

Figure 16 Rapid Impact Compaction (a) principle and (b) used for a reclamation project at Jurong Island

3.2.2 Dynamic compaction

Dynamic compaction is a densification technique involving lifting and dropping of heavy weight or pounder onto the ground surface in a repetitive manner. The impact force generated from the heavy weights rearranges the soil particles to a denser state. Figure 17 shows a typical schematic of a dynamic compaction equipment with a pounder attached to a crane. The efficiency of the method and depth of compaction are affected by various factors such as weight of impact, drop height, number of drops, spacing between drops, surface area of the pounder and drop intervals (Mayne et al. 1984).



(a) (b)

Figure 17 Dynamic compaction (a) principle and (b) compaction machine used for the Changi East land reclamation project

The effective depth of DC may be estimated by the following equation (Chu et al. 2009c):

$$D = Cn\sqrt{(WH)} \qquad (8)$$

where: C is a coefficient for the type of drop. Its value is given in Table 7. n is a correction factor. n = 0.35 – 0.6 for sands as suggested by Lukas (1995). For the Changi East land reclamation projects, some n values have been back calculated as n* using the influence depth as determined using CPT. This n* is equivalent to Cn for C = 0.89. Thus, the n values can be calculated as given in Table 8. It can be seen these values fall between the values suggested by Lukas (1995).

Table 7 Coefficient value of C in Equation (8)

Drop method	Free drop	Rig drop	Mechanical winch	Hydraulic winch	Double hydraulic winch
C	1.0	0.89	0.75	0.64	0.5

Table 8 Back calculated n value based on influence depth determined using CPT (after Choa et al. 1997)

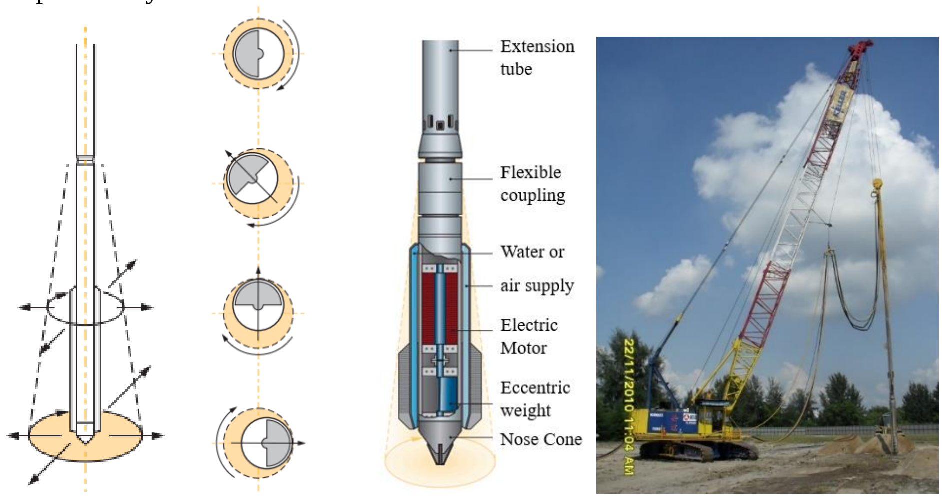
Pounder mass (t)	15	14	23	23
Drop height (m)	20	20	12.5	25
Pounder surface area (m^2)	3.87	2.25	5.5	5.5
Energy (t-m)	300	280	287.5	575
Influence depth (m)	7.5	7	6	8
n* = Cn (back calculated)	0.433	0.418	0.354	0.334
n (in Eq. 8)	0.487	0.460	0.398	0.375

3.2.3 *Vibro compaction*

Vibro compaction is a deep compaction technique, adopted for densifying cohesionless soils using the depth vibrator typically suspended from a crane, by means of horizontal vibrations during the withdrawal. Figure 18a-b shows the principle of vibro compaction and depth vibrator. The ground is levelled by adding additional granular backfill material during the compaction process. Figure 18c shows vibro compaction used at a reclaimed land site at Tuas, Singapore.

The feasibility of the vibro compaction technique depends on the soil grading and fines content present in the soil. In general, well graded granular soils with fines content below 10% are ideally suited for vibro compaction. Granular soils possessing coefficient of uniformity ≤ 2 may have difficulty in compaction, while the presence of more than 10% fines may inhibit the compaction process (Chu et al. 2009c; Kirsch and Kirsch, 2017).

In order to verify the soil conditions and determine the depth of treatment, Cone Penetration Tests (CPTs) are performed before vibro compaction. The spacing between the vibro compaction points depends on the structural loading and type of vibrator used. Usually, a trial of vibro compaction is carried out to check the ground compactability.



(a) (b) (c)

Figure 18 Schematics showing (a) Principle of vibro compaction, (b) Depth vibrator (source: Keller Group) (c) Vibro Compaction done for development on reclaimed land at Tuas

The cost comparison of the three compaction methods are illustrated schematically in Fig. 19. The DC is getting more expensive with increasing in treatment depth. The cost of vibro compaction, on the other hand, is getting less with increasing in treatment depth. Therefore, it will be more cost-effective to find out an optimized depth where DC can be applied for the layer above this depth and vibro for the layer below this depth.

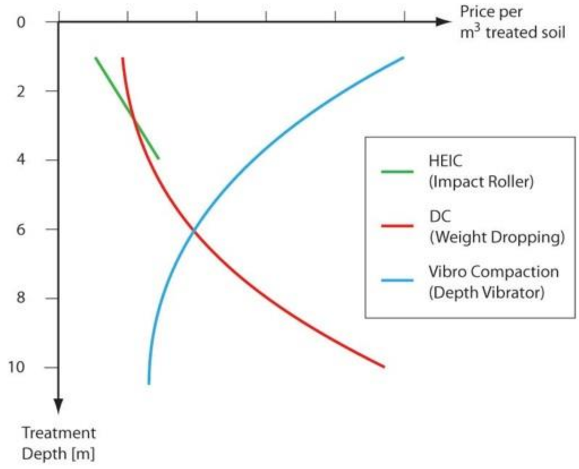


Figure 19 Cost comparison of three different compaction methods (source: http://www.groundimprovement.ch/Ground_Improvement_Solutions/Impact_Compaction.html)

3.3 Applications in Singapore

Both DC and vibro compaction have been used in the Changi airport reclamation and Changi East reclamation project (Choa et al. 1980; Bo et al. 2005). Rapid Impact Compaction has been recently used for compaction works at Jurong Island. Vibro compaction has also been extensively implemented in Singapore for reclamation projects such as Tekong reclamation and Jurong reclamation; airports such as Changi Airport Runway No. 3 and Taxiway; highways and chemical plant in Jurong Island (Raju and Sondermann, 2005) and recently in Pasir Panjang Container Terminals, Tuas extension and development projects, Changi Airport Terminal 5, Jurong extension project and Changi East airfields. In Singapore, the total volume of Vibro Compaction works has exceeded 300 million m^3. Table 9 summarizes various densification techniques employed for some major reclamation projects in Singapore.

Table 9 Densification techniques employed for some major reclamation projects in Singapore

Reclamation Project	Densification methods	References
Changi airport reclamation	Dynamic compaction	Choa (1980)
Changi East reclamation	Dynamic compaction, Muller resonance compaction, Vibro compaction	Bo et al. (2005)
Tekong reclamation	Land and offshore vibro compaction	Raju and Sondermann (2005), Raju (2009)
Jurong reclamation	Vibro compaction	Raju and Sondermann (2005)

DC or vibro compaction have also been used for port construction. For example, vibro compaction have been used to densify sand fills used to support the caisson walls as well as the sad fills for land reclamation in Pasir Panjang Container Terminal (Raju and Sondermann, 2005). Vibro compaction was also used for the Tuas Port Container Terminal Development. Other applications of densification techniques include slope stabilization for peripheral bunds (Fig. 20a), foundations for heavy and settlement-sensitive structures (Figs. 20b-c), runways and highways (Figs. 20c-d), sand key compaction for caissons (Fig. 20e), as well as prevention of liquefaction.

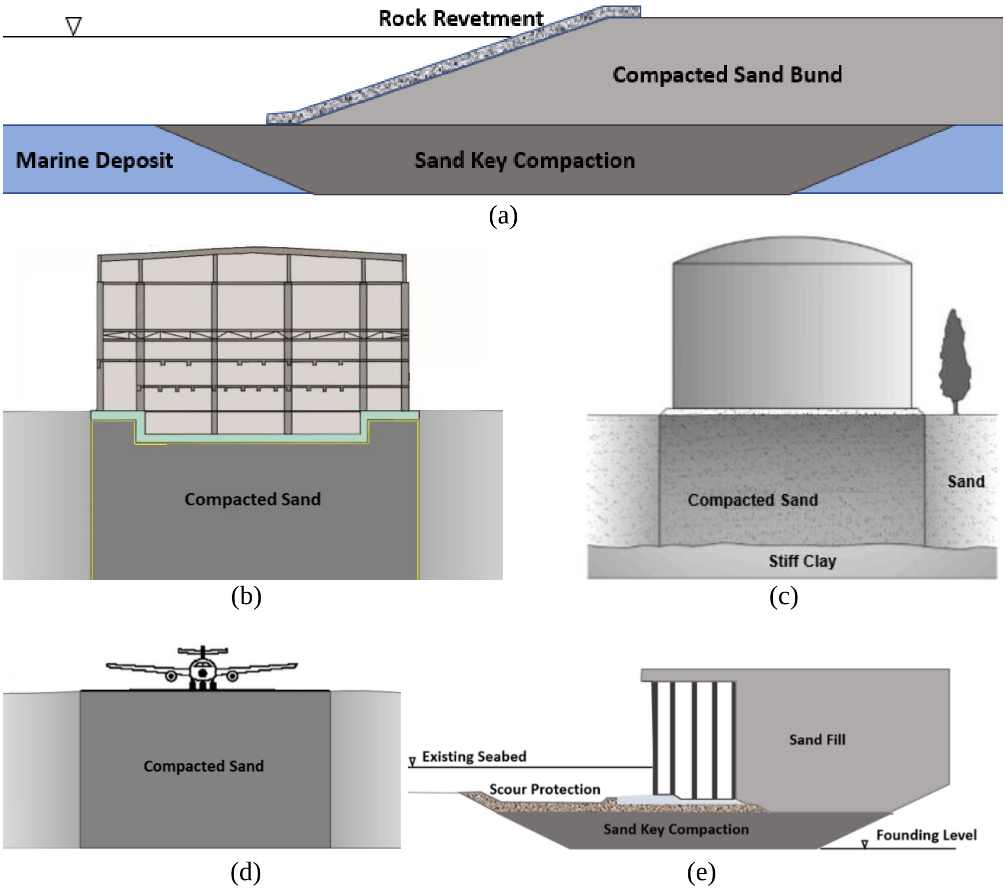
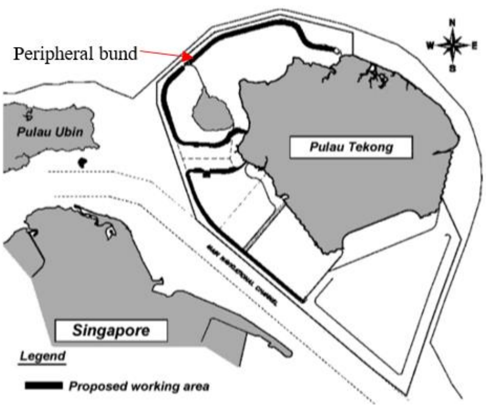


Figure 20 Application of densification techniques in (a) Stability of sand bunds, (b) foundation for large workshops, (c) foundations for storage tanks, (d) foundation for Airport Runways, (e) sand key compaction for caissons, (modified after Raju, 2009; Leong et al. 2012, He et al. 2016; Daramalinggam et al. 2009; Raju and Sondermann, 2005)

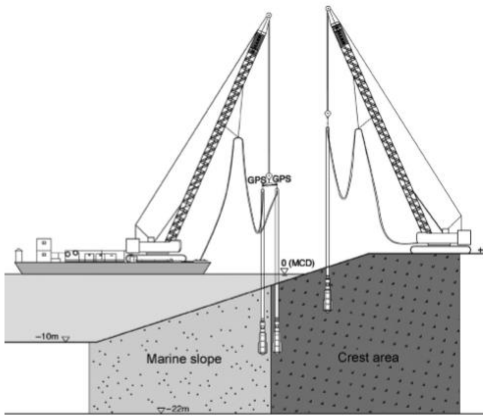
3.4 Case studies

3.4.1 Tekong reclamation project (Raju and Sondermann, 2005)

By 2005, the Tekong Island consisted of over 1.3 million m^2 of reclaimed land. In Fig. 21a, the dark line along the periphery indicates the perimeter bund where the compaction works were carried out. Figure 21b shows a typical cross section through the perimeter bund showing the regions of land and marine compaction. The purpose of the compaction works was to ensure slope stability and reduce settlements without causing any damage to the rock revetment which was infilled with cement mortar. Both the sandkey and the reclamation slope were compacted using vibro compaction. A total volume of 17.5 million cubic meters of sand fill was compacted using both land and marine equipment. The maximum depth of compaction at the crest of the perimeter bund from a land rig was about 43 m. Figure 22a shows both land and marine operations being carried out for the perimeter bund and human-made beaches. Post-cone penetration tests (CPTs) were carried out to assess the degree of compaction. Post-CPTs were performed at every 50 m interval of sand bund in both land and marine compaction regions and showed the degree of compaction achieved after vibro-compaction was satisfactory (Fig. 22b).



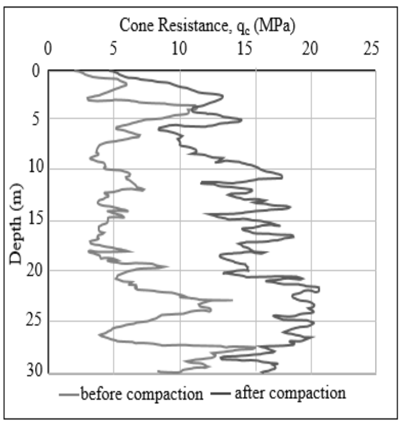
(a)



(b)

Figure 21 (a) Plan view showing reclamation area and peripheral bund (b) Typical cross section of peripheral bund showing regions for land and marine compaction (modified after Raju and Sondermann, 2005)

(a)



(b)

Figure 22 (a) Compaction being done for land and marine regions (b) A representative set of results obtained for pre- and post-CPTs (re-drawn after Raju and Sondermann, 2005)

3.4.2 Chemical plant in Jurong Island (Leong et al. 2012)

The specific chemical plant cited here houses various chemical processing facilities, pipe racks, towers, tanks and low rise reinforced concrete structures within a land area of 46,000 m^2. The depth of the loose sand reclaimed fill was about 20 m. The fill was vibro compacted to support moderately loaded plant facilities. An approximate area of 31,400 m^2 was improved by vibro compaction within the project site.

One of the buildings was founded on a 47.4 m ×40.1 m raft with a thickness of more than 1 m, with an average design load of 107 kPa and the maximum allowable settlement of 50 mm. Vibro compaction was required to densify the sand fill determined based on soil investigation comprised of 54 boreholes and 44 pre-CPTs. Vibro-compaction works for a total of 3,600 m^2 area were completed within the period of 20 days as compared to 3 mths if piling works were adopted. The quality of the compaction was evaluated by a total of 135 post-CPTs. A comparison of pre-CPT and post-CPT results is shown in Fig. 23. It can be seen that the cone resistance within the sand layer increased significantly to a minimum value of 15 MPa. As shown in Fig. 24, the maximum settlement recorded by the settlement markers over a period of 1 year was 21 mm which was less than the allowable settlement of 50 mm.

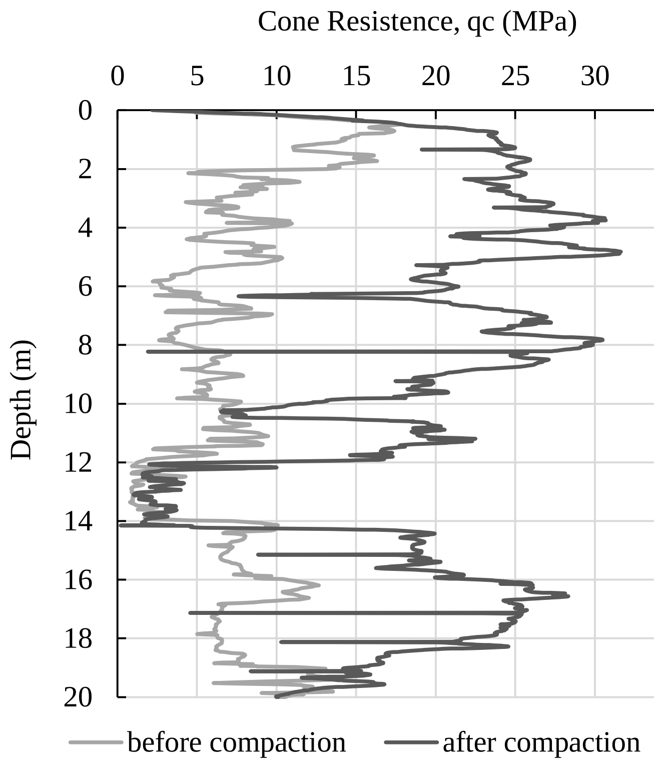


Figure 23 Pre-CPT and Post-CPT results taken before and after compaction of fill

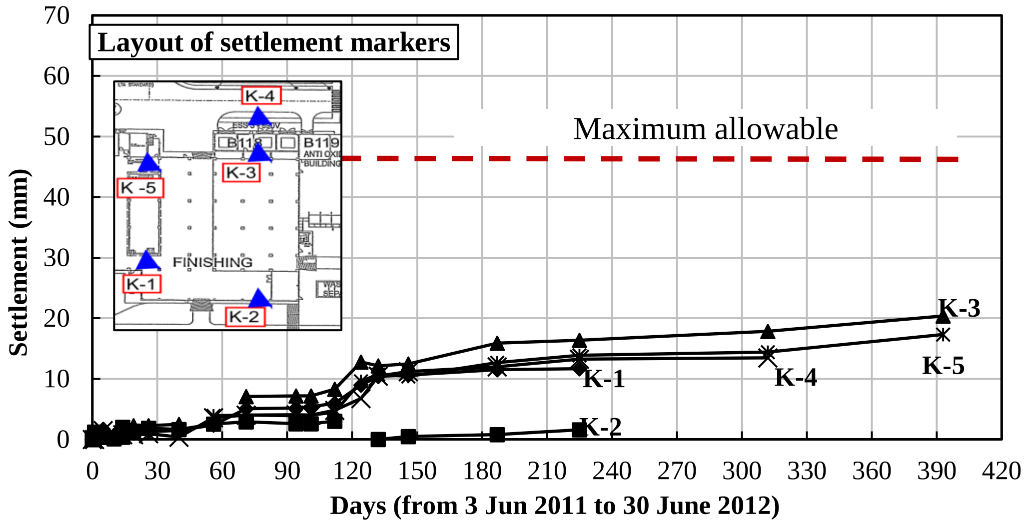


Figure 24 Settlement monitoring data for building (modified after Leong et al. 2012)

4 VIBRO REPLACEMENT AND DYNAMIC REPLACEMENT

4.1 Introduction

Vibro replacement (VR) is an extension of the vibro compaction method described in Section 3. In this method, granular materials are fed into a borehole created using a vibrator and compacted using the same vibrator to form rigid columns. It is also called the stone column method when stones are used. In theory, the method can be applied to all types of soils. However, it is mainly used to improve soft or weak soils. The common construction methods for stone columns include (1) wet top feed method; (2) dry bottom feed method; and (3) offshore bottom feed method.

Dynamic replacement (DR) is an extension of the dynamic compaction method described in Section 3. DR columns are formed by placing a blanket of aggregate over the site, and driving the aggregate into the soil by dropping a 15 to 30 ton pounder from heights ranging from 9 to 36 m, an operation similar to dynamic compaction. The method improves the strength of saturated cohesive soils and soft organic soils, when dynamic compaction is not effective due to the high fines content of the in-situ soils. The application of DR in Singapore is not frequently in the recent years.

4.2 Vibro replacement

One of the most commonly used VR technique is Stone columns which have experienced substantial progress in recent years due to the improvement in the equipment and monitoring systems. The wet top feed method has gradually being replaced by the dry bottom feed method. This is partially due to undesirable effect of water and flooding of the working surface. The dry bottom feed method is illustrated in Fig. 25. The machine used has penetration capacities of 10 to 16 m and is equipped with continuous stone feeding systems mounted on self-erecting crawlers (Fig. 26a). Furthermore those rigs are capable of pulling down vertically to ensure the quality of the continuous columns. For deeper columns, a free hanging system as shown in Fig. 26b could be used for greater depth and offshore works.

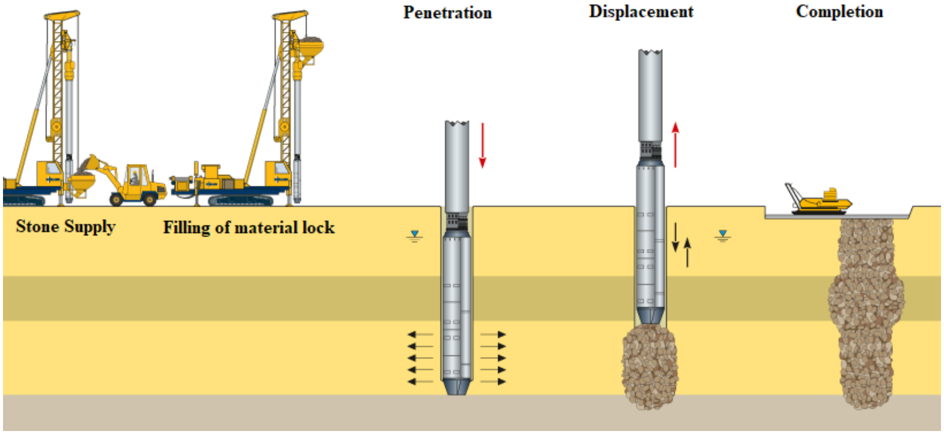


Figure 25. Dry bottom feed vibro replacement method (source: Keller group)

(a) (b)

Figure 26. Vibro replacement system (a) dry bottom feed system and (b) a free hanging installation system for offshore works

4.2.1 Applications of vibro replacement in Singapore

Vibro replacement (VR) has been used for many projects in Singapore to stabilize slopes for road or rail embankments (Fig. 27a), to control settlements for highways and expressways (Fig. 27b), to control settlements under settle-sensitive structures (Fig. 27c), to strengthen soft clays under sand bunds for reclamation projects (Fig. 27d), etc. Table 10 summarizes various projects in Singapore which have involved VR techniques.

Table 10 Various applications of vibro replacement techniques in Singapore

Application	Projects	References
Slope stability	Offshore Reclamation slope, Tekong Reclamation	Leong and Raju (2007)

Foundation	Seraya Island Tank Farm	Raju et al. (2016)
	Oil tanks at Jurong Island	Daramalinggam et al. (2009)
	Residential development driveways, Kallang-Whampoa area	Daramalinggam and Prasad (2016)
	Oil/Chemical tanks, offshore Island	He et al. (2015; 2016)

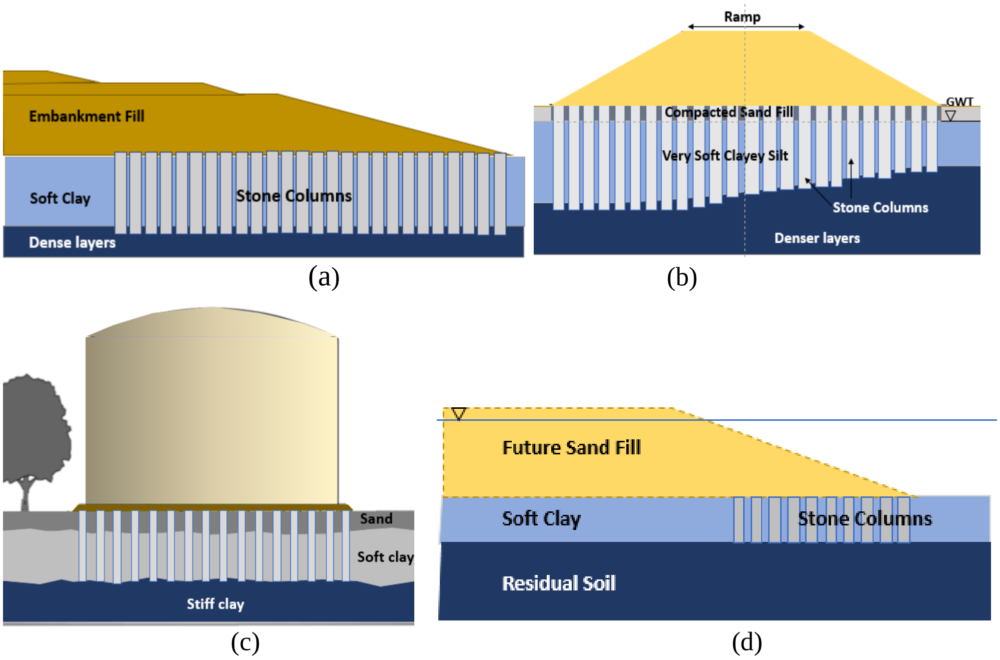


Figure 27 Application of vibro replacement or stone columns techniques for (a) expressways (b) highways (c) large storage tanks and (d) offshore reclamation slopes

4.2.2 Case studies

The first case is the installation of stone columns into marine clay for the stability of an offshore dike in Singapore as shown in Fig. 28. The system shown in Fig. 26b was used for this project (Leong and Raju 2007). In this bottom feed system, stones were pumped through a 200 mm diameter hose to the top of the vibrator using high velocity water. Stone columns were installed under marine conditions, up to 28 m below sea level. Stone-columns of diameter 1.1 m with square-grid spacing of 1.78 m and 30% replacement ratio were adopted. Stones of size varying from 10 to 38 mm were used for this project. For details of case-histories related to each application, see Raju et al. (2004); Raju and Sondermann (2005); Leong and Raju (2007).

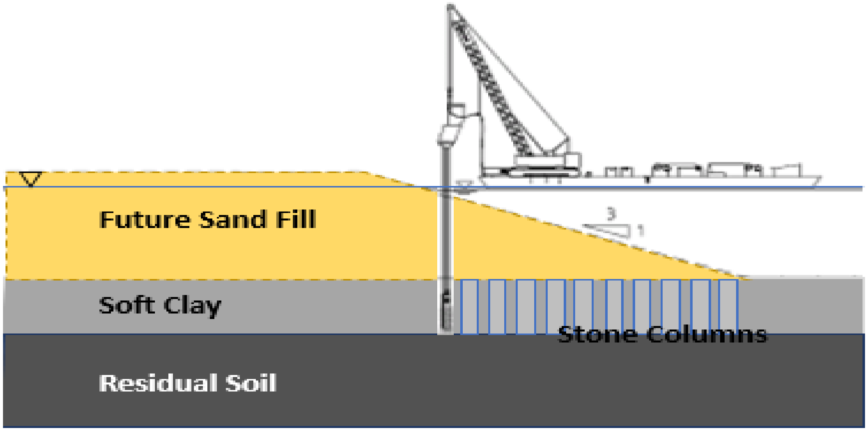


Figure 28 Stone column installation from a barge (modified after Leong and Raju 2007)

Another case history is the use of vibro compaction for steel tanks in an offshore island, Singapore. As a part of expansion of an existing tank terminal, new tanks with diameters ranging from 36 to 61 m and height of about 21.6 m had to be constructed. Soil investigations by boreholes and Cone Penetration Tests indicated that the top 4-5 m layer consists of reclaimed sand fill, followed by soft to firm silty clay mixed with seashells. Stiff to very stiff clay was found after a depth of about 12 m. The allowable tank edge settlement was 150 mm during hydrotest and 75 mm post hydrotest. The settlement markers were placed along the circumference of all tanks as per API Standard 653 (2009) before the start of hydrotest.

A combination of vibro compaction for the sand layer and vibro replacement for the soft to firm seabed clays below the sand layer were proposed as a ground improvement solution as shown in Fig. 29. Stone columns of diameter varying from 0.8 to 1.0 m with grid spacing of 2.0 to 2.5 m were installed depending on soil conditions.

A hydrotest was carried out after the ground improvement and construction of the tank. The observed settlements at the edge around the tank were plotted against time for all settlement markers during the hydrostatic test in Fig. 30 for a tank of 44.2 m diameter. The maximum and minimum edge settlements across all settlement markers were found to be in the range of 57–91 mm and 29–69 mm, respectively. The average settlement varied between 44 and 74 mm which was well below the permissible limit of 150 mm during hydrostatic test. The predicted settlements for post hydrotest were estimated to be in the range of 45–71 mm which were less than the maximum allowable settlement of 75 mm. Hence, the measured settlements were well within the design prediction and the tank performance was found to be satisfactory per API Standard 653.

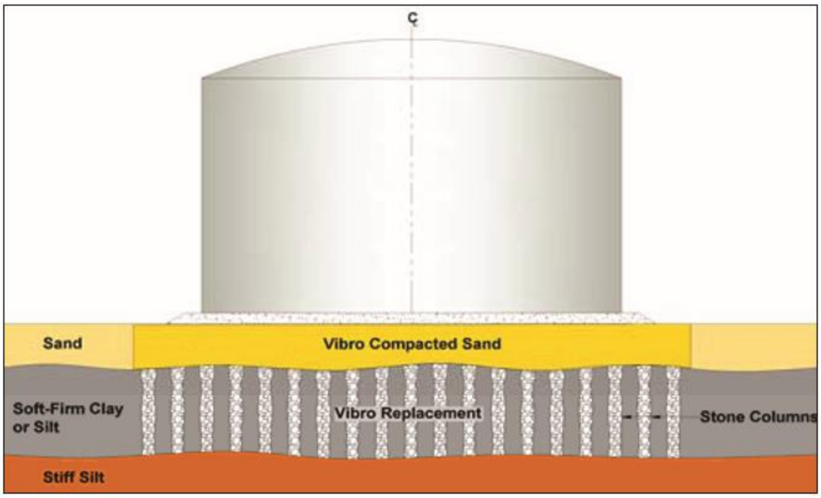


Figure 29 Ground improvement scheme using vibro techniques applied at the tank terminal expansion (after He et al. 2015)

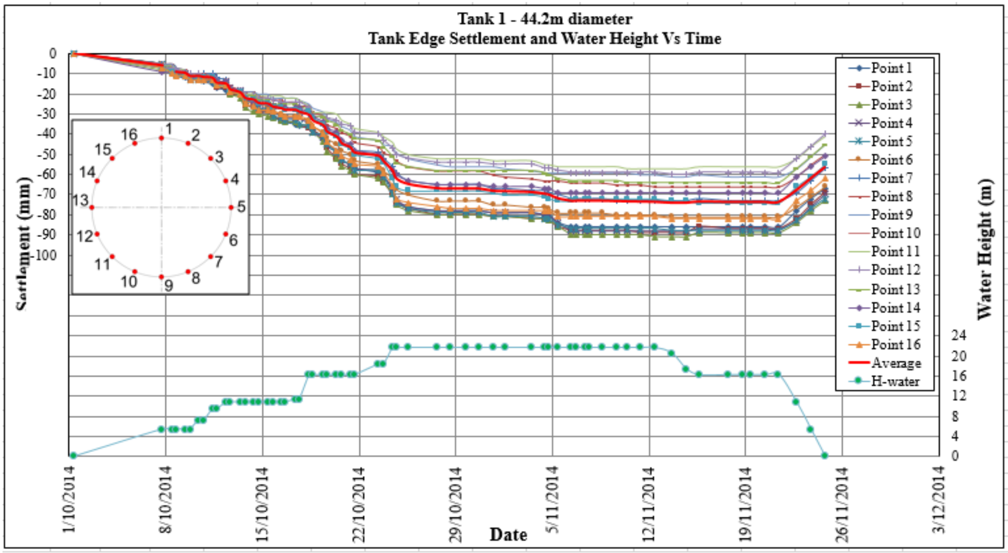
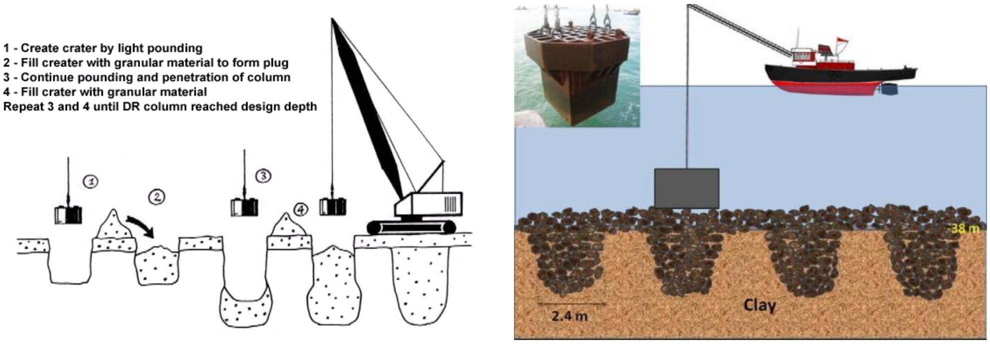


Figure 30 Plot of observed tank edge settlements vs time and water height vs time during hydrostatic test for a 44.2m diameter tank (after He et al. 2015)

4.3 Dynamic replacement

Dynamic replacement (DR) method has also been used in Singapore although less frequent than the use of vibro replacement. The method improves the strength of saturated cohesive soils and soft organic soils, where dynamic compaction becomes less effective due to the high fines content in the soils. The DR process is illustrated in Fig. 31a. It starts out by producing a crater with light pounding. The craters are then backfilled with granular materials such as aggregates, stone, gravel or rocks that will lock together under subsequent heavy pounding. This pounding process is repeated until a noticeable decrease in crater formation occurs. Typically the diameter of the DR columns ranges from 2.5 to 5.0 m and the depth is up to 8 m. Case histories of ground improvement projects using the DR method include the improvement of marine clay pits on reclaimed land at Pasir Ris, Singapore (Wei et al. 1993); the improvement of clayey silt for two large oil tanks with diameter up to 69 m in Vietnam (Ong et al. 2007); and a land fill project at Tampines, Singapore (Keller project 2012). The DR method is normally adopted on land. In a recent port project in Singapore, a pounder as shown in Fig. 31b was used for offshore compaction to improve the shear resistance of soft seabed.



(a) (b)

Figure 31. (a) Dynamic replacement process; and (b) Application of offshore dynamic replacement in Singapore

It should be mentioned that a dynamic replacement and mixing (DRM) method was also developed by Ramaswamy et al. (1979) and Lo et al. (1990a, 1990b) for the treatment of peat or other types of highly organic clay. This method can be considered as an extension of the DR method. It consists of DR and an additional step to compact the DR columns installed in the clay as shown schematically in Fig. 32. DR columns are formed by tamping using low energy blows (Fig. 32a). After the DR columns are formed (Fig. 32b), tamping using sufficiently high compacting energy per blow of a pounder is applied to cause jets of sand to be ejected from columns into the peaty clay surround by a process resembling claquage (Fig. 32c). The rationale for this method is to disrupt the in-situ soil fabric to such a degree that its inherent secondary compression characteristics will effectively be nullified and thus the secondary compression would be controlled under the future work load. A full scale field trial was carried out in 1983 at a MRT depot site in Singapore to evaluate the feasibility of this method (Lo et al., 1990a).

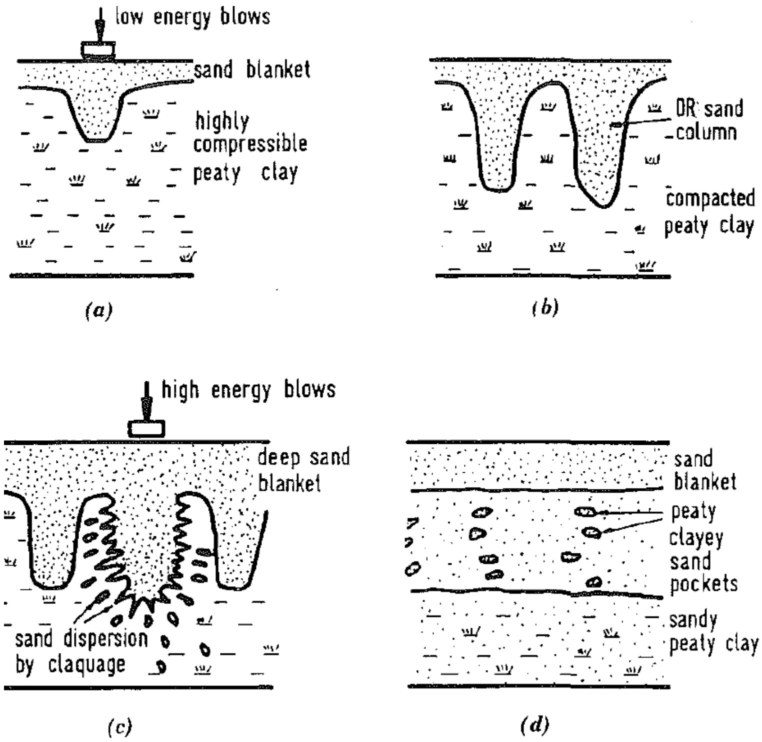


Figure 32. Mechanisms of DRM: (a) Formation of DR column; (b) Subsoil profile after DR treatment; (c) DRM Process; (d) Subsoil profile after DRM treatment (after Lo et al. 1990a)

5 JET GROUTING AND DEEP SOIL MIXING

Jet grouting and deep cement mixing are other ground improvement methods that are commonly adopted in Singapore for MRT, excavation works and other projects. Jet grouting in principle is a semi or complete replacement method and thus can be applied to a wide range of problems including very difficult ground conditions. Deep soil mixing (DSM) method is to mix soil with cement, lime or other binders in-situ using a specially made machine. This method was developed in Japan and in the Scandinavian countries independently in the 1970s and thus has been called in different names, but commonly referred to as deep soil mixing (DSM) or deep cement mixing (DCM). There are generally two installation methods, the dry mixing and wet mixing. The wet mixing method is commonly adopted in Singapore.

5.1 Jet Grouting

5.1.1 Introduction

Jet grouting is a ground improvement technique used for strengthening the in-situ soil, providing structural rigidity and cut-off from water seepage. This method involves applying high-pressure jets of water or/and air with grout which strengthens the existing ground (Essler and Yoshida, 2004). Fig. 33 shows the several types of jet grouting systems. Double jet system can result in larger diameter jet grouting piles, however has a limitation that a part of grout is often transported back through the hose along with spoil. This can be overcome by use of triple jet systems in which water jet is used with the compressed air. Fig. 34 shows a typical set-up and working sequence for jet grouting technique. In Singapore, double and triple fluid systems are most commonly used, mainly to treat Singapore marine clay having water content of 55 to 70%, plastic limit of 38 to 55%, liquid limit of 65 to 85% and shear strength of 10 to 40 kPa (Ganeshan and Yng, 2009). Grouting slabs/piles may be designed as horizontal reinforcements when installed as improved soil layer at the base of excavation as well as vertical reinforcements when installed under structures as bearing elements. Jet grouting is often applied as a temporary support during tunneling in difficult ground conditions. Fig. 35a-c show a jet grouting equipment and typical double fluid system used for a MRT project.

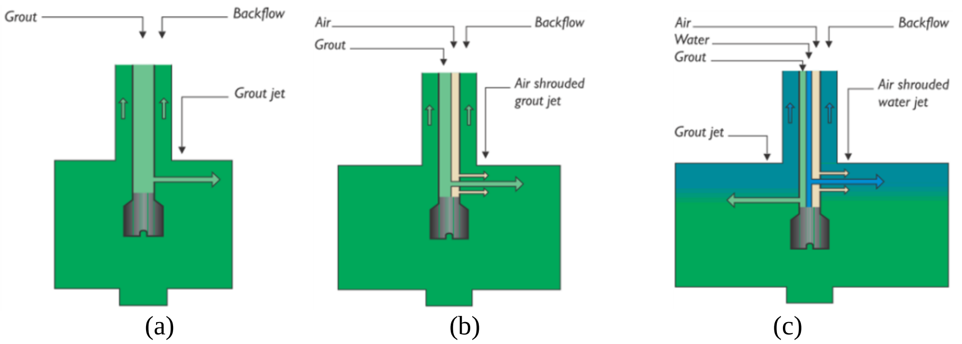


Figure 33 Schematics showing types of jet grouting systems (a) Single fluid (b) Double fluid (c) Triple fluid (source: Keller group)

Jet grouting method has undergone a number of modifications and expansions as described by Chu et al. (2009c). These include superjet grouting (Burke et al., 2000), X-jet grouting (Welsh and Burke, 2000), Rasjet methods (Osborne and Ng, 2008). Some of those methods can be used to create large diameter columns. It is claimed that the superjet method can create 5 m diameter columns with strength of 6 times stronger that those of normal jet grouting columns (Hashimoto and Liu, 2011). Standards for jet grouting including BS EN12716 (2001) - Execution of Special Geotechnical Work –Jet Grouting have been established.

Another interesting development in the jet grouting method is the so-called MultiFan method developed by Sanshin Corporation in Japan (Shinsaka et al. 2017). By alternation of slow and fast rotational speeds, as shown in Fig. 36a, where Vr1 and Vr2 are the rotational speeds through the fast and slow sectors respectively, an elliptical column can be formed. Specifically, when the injection nozzles are oriented towards the designed longer axis of the ellipse (or column), the rotational speed of the jet grout rods is reduced in order for the pressurized fluids to act for a longer time onto the soils (slow sector). A grouting column formed is shown in Fig. 36b. This method has not been used in Singapore.

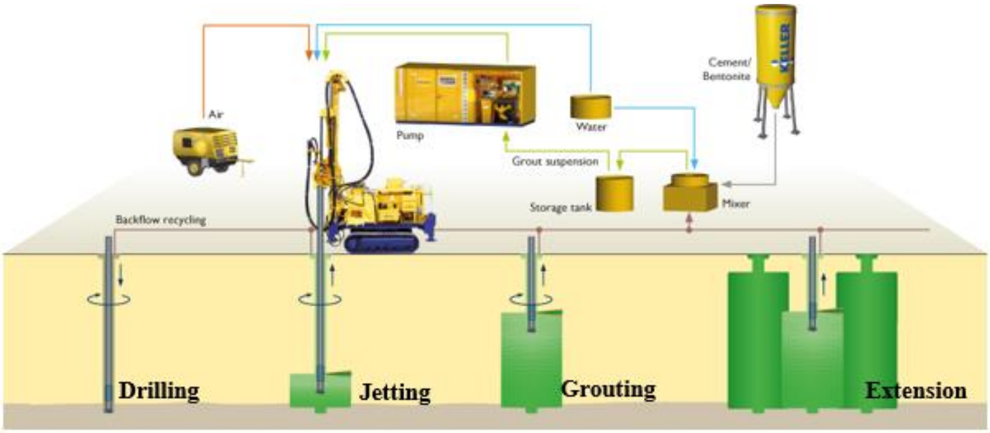
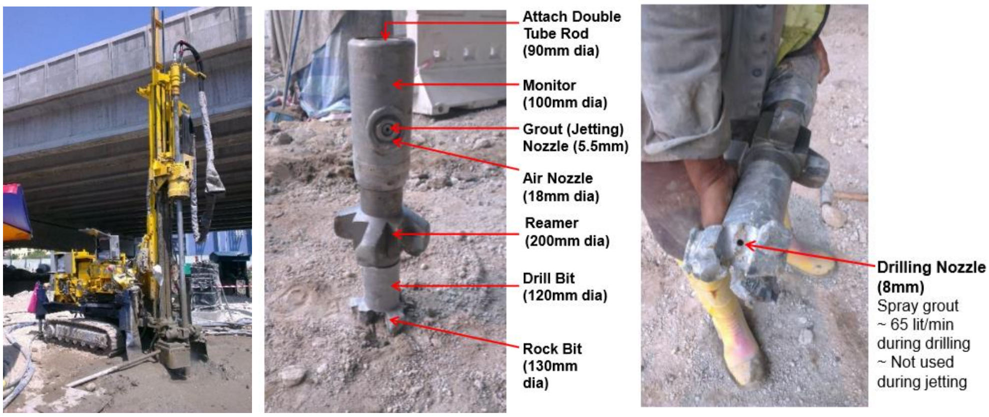


Figure 34 A typical jet grouting set-up and working procedure (source: Keller group)



(a) (b) (c)

Figure 35 (a) Jet Grouting done for MRT project at Malaysia (b)-(c) Double fluid type jet grouting systems (source: Keller ASEAN)

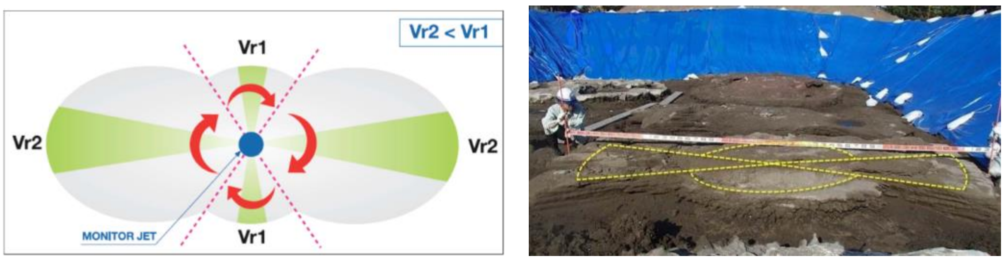


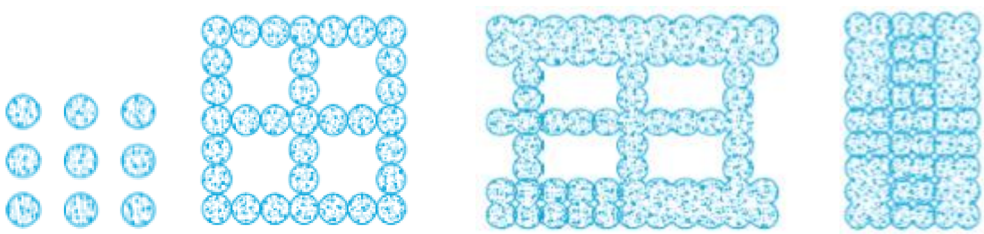
Figure 36. The MultiFan jet grouting (a) principle (b) Column formed in the field (after Shinsaka et al. 2017).

5.2 Deep soil mixing

5.2.1 Introduction

Deep soil mixing (DSM) involves construction of cement/lime admixed columns with the help of hollow rotating shafts along with cutting and mixing tools for cutting the ground and simultaneously injecting and mixing the admixtures with/without water with the in-situ soil. DSM can be used for waterfront and marine structures (quay walls), foundations and underground structures, earth retaining structures, seepage cut-off walls, liquefaction mitigation, etc. (Porbaha et al. 1998). DSM columns may be arranged in various column patterns depending on the nature of application, strength requirements, site conditions and cost of treatment (Fig. 37). Figure 38 shows the typical schematics of twin-shaft deep soil mixing equipment. Comprehensive reviews and descriptions of the various methods of deep mixing and applications have been given by Terashi (2003), Topolnicki (2004), Larsson (2005), Essler and Kitazume (2008), and Chu et al. (2009). Standards such as BS EN 14679 (2005) for deep mixing have been established. The recent developments have mainly taken place in the optimisation of the process and the optimisation of tools for mass production.

In Singapore, deep soil mixing has become a popular alternative to jet grouting in recent years. This is because jet grouting involves injecting water, air along with grout at high pressure, this may cause expansion in the in-situ soil. DSM causes less expansion when compared to jet grouting. Another advantage of DSM is that, jet grouting produces much higher amount slime as an industrial waste which has to be properly disposed (Tan et al. 2002). However, jet grouting can be adopted in areas with height restriction and with underground utilities, due to reduced height of equipment and choice of inclined application. Cement is preferred over lime due to problems in storage of unslaked lime due to hot and humid weather (Tan et al. 2002). Figure 38a and 39b show the jet grouting and deep mixing equipment used for various excavation projects in Singapore and Malaysia.



(a) (b) (c) (d)

Figure 37 Different types of columnar structure constructed with DSM (a) isolated columns, (b) grid, (c) grid with overlapping columns, and (d) block with overlapping columns (modified after Topolnicki, 2004)

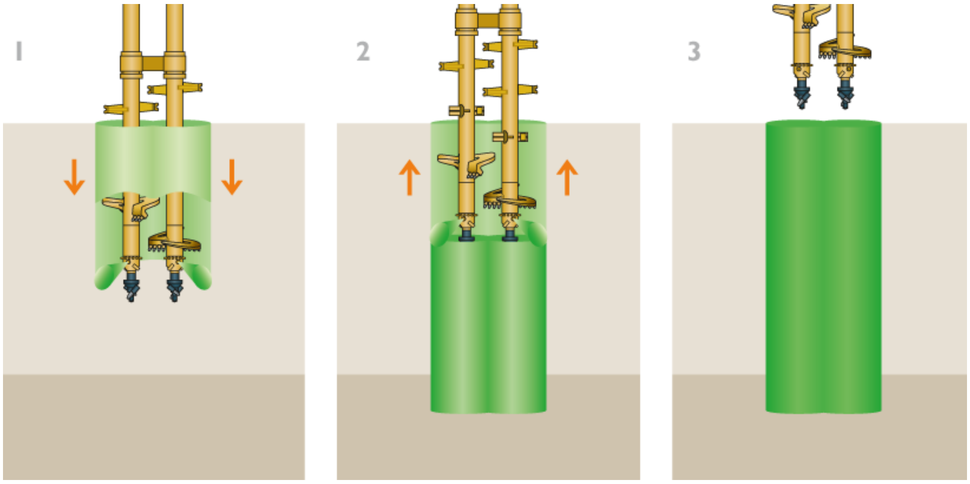


Figure 38 Schematics showing treated columns constructed by twin-shaft Deep soil mixing equipment (source: Keller group)

(a) (b)

Figure 39 Dual shaft deep mixing rigs used for tunneling project at Bedok South, Singapore (source: Keller ASEAN)

5.2.2 Wet speed-mixing

One relatively new DSM method is the so-called wet speed mixing (WSM). A powerful drill advances a mixing tool as binder slurry is pumped through the connecting drill steel, mixing the soil to the target depth. Additional mixing of the soil is completed as the tool is withdrawn to the surface. Mass wet soil mixing, or mass stabilization, is performed with a horizontal axis rotary mixing tool at the end of a track hoe arm. The binder slurry is injected through a feed pipe attached to the arm. This method combines the use of DSM and jet grouting mechanisms and offers advantages such as quick, vibration free, and low noise. This method can be used nearly for any soil types ranging from soft clay to sandy soil including organic soil. Some applications of the WSM method in Singapore involving ground improvement for peaty or soft soils for 6 to 35 storey residential buildings has been reported by Arulanantham and Yogarajah (2016). WSM methods have also been used for several MRT constructions including the Marina Bay Station and Tunnels for Thomson-East Coast Line and the Rochor and Little India Stations and Tunnels for Downtown Line. The operational parameters for WSM as described in Arulanantham and Yogarajah (2016) and by Zhao Yang Geotechnic are summarized in Table 11.

Table 11 Operational parameters for WSM (modified from Arulanantham and Yogarajah 2016)

Parameters	Unit	Values
Normal diameter	m	0.6 to 2.6
Treatment depth	m	Up to 53 m
Spacing C/C	m	Depend on design (1.5 to 2.0 m)
Mechanical mixing grout pressure	MPa	5 to 20
Diameter of mixing monitoring head	mm	Depend on design
Rotation per min penetration	RPM	30 to 90
Discharge rate of cement grout	L/meter	275 ± 10
Grout mix ratio	-	300 kg/m^3
Grout specific gravity	-	1.48 ± 0.03
Design undrained shear strength	kPa	> 500
Design Young’s modulus	MPa	150

5.3 Applications of jet grouting and deep soil mixing in Singapore

Jet grouting has been used for controlling settlements due to tunneling (Fig. 40a), as a horizontal/passive support in deep excavation (Fig. 40b), preventing basal heave failure (Fig. 40c), controlling ground movement to protect adjacent structures, etc. (Ganeshan and Yng 2009). Deep soil mixing (DSM) has also been used in similar applications, for example to control movements during deep excavation for a tunnel (Fig. 41a), as sacrificial DSM slabs for construction of launching shaft (Fig. 41b), to provide face stability and control surface settlements during tunnel break-in and break-through (Fig. 41b), for slope stability of temporary cut-slopes (Fig. 41c), as a retaining structure (Fig. 41d). DSM and jet grouting are often used together, for example, to provide support in Earth Retaining and Stabilizing System (ERSS), tunnel break-in/ break-out where some areas have overhead restrictions or require inclined grouting.

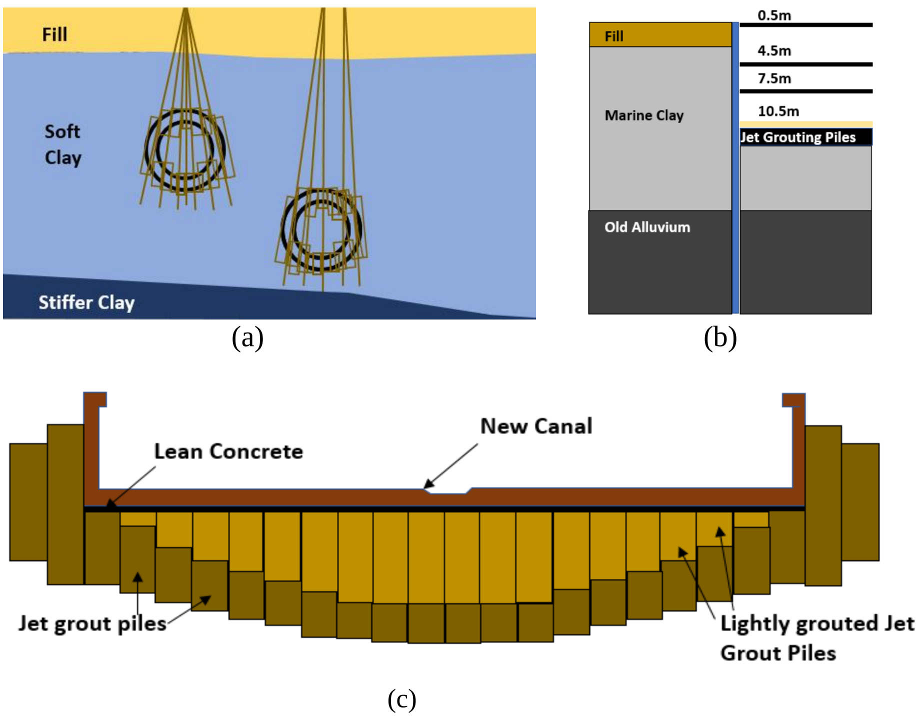
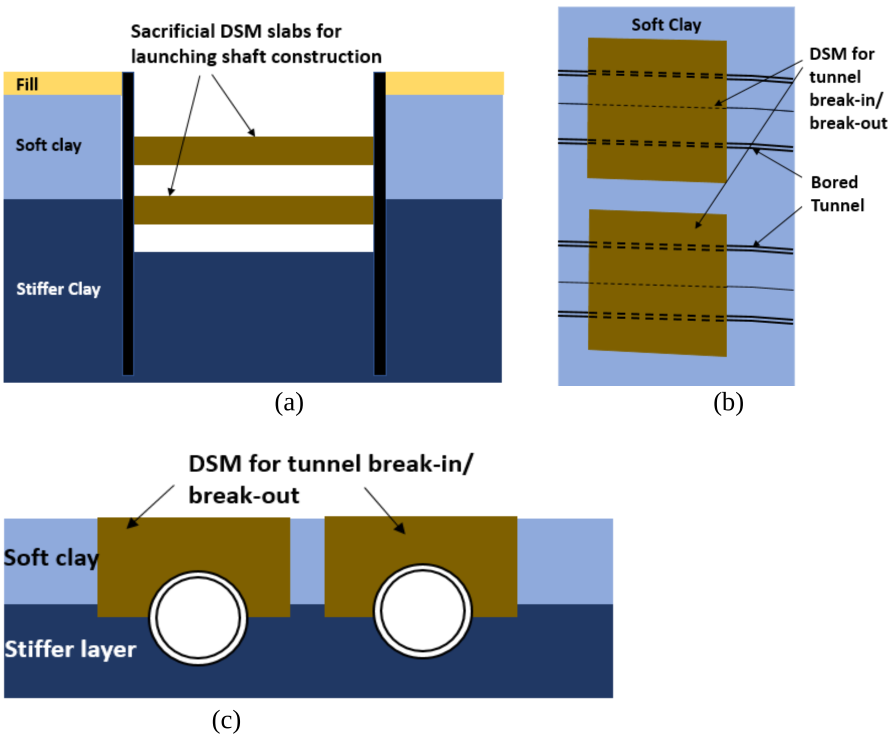
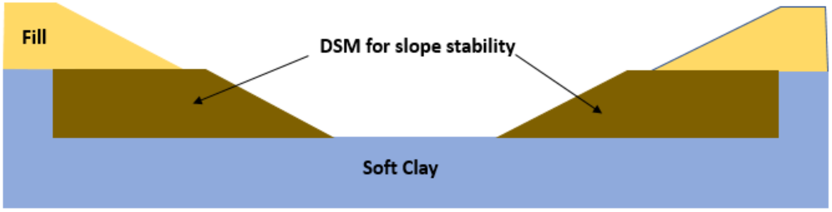
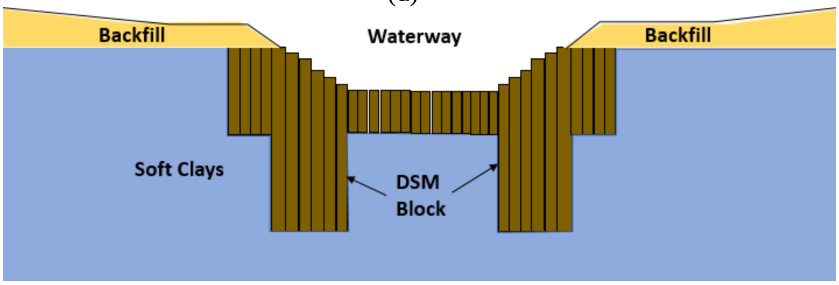


Figure 40 Applications of jet grouting in Singapore, (a) Controlling settlements due to tunneling, (b) Enhancing passive resistance during deep excavation between opposite walls, and (c) Preventing basal heave failure for canal (modified after Ganeshan and Yng, 2009)





(d)



(e)

Figure 41 Applications of deep soil mixing in Singapore, (a) as sacrificial slabs for launching shaft, (b)-(c) interface support for TBM break-in/ break-out (d) Slope stability to control erosion, (d) DSM blocks as retaining structure for waterway

Table 12 summarizes applications of jet grouting and DSM in various projects of Singapore. The total deep soil mixing and grouting works in years 2015-2016 was over 0.8million m^3 mainly consisting of projects for Thomson East Coast Line and Changi Water Reclamation Plant. In 2017, the volume of cement treatment works consists mainly of Tekong Polder, North-South Corridor and Circle Line 6 and is estimated to be over 1.8 million m^3.

Apart from these techniques, hybrid techniques are also being used for various projects. For example, in new Nicoll Highway MRT station, Circle Line Project and new Singapore Management University, a modified deep mixing method called Jet Mechanical Mixing (JMM) was adopted, in which the inner part of the column is made by mechanical mixing while outer part of the column is created by high pressure jet grouting technique (Page et al. 2006; Chen et al. 2011). For the reconstruction of the Nicoll highway station, JMM or the so-called RASJET method was adopted for deep excavation in soft Singapore marine clay (Osborne and Ng 2008). JMM is a combination of soil mixing and jet grouting that produces overlapping columns with an internal column of mixed soil by the auger and an external column created by a slurry jet into the in-situ soil. The JMM machine is shown in Fig. 42. The process of forming the columns is similar to the method of forming jet grouting columns with the addition of dual and counter rotation mixing blades on the drill rod to ensure intensive soil mixing. To install a JMM column, the auger is first drilled to the base level of the JMM column with water injection, and withdrawn to the top level of the JMM column with mechanical mixing without any injection. It then descends with slurry injection and mechanical mixing to form the internal soil mixing column up to base level. After that, it ascends with jetting to form the external jet grouting perimeter.

Table 12 Application of Jet Grouting and Deep Soil Mixing in various projects of Singapore

Technique	Application	Projects	References
Jet grouting	Deep/Basement excavation	Newton MRT Station	Gaba (1990)
		Singapore Arts Center, Bugis	Sugawara et al. (1996)
		Singapore Post Centre	Poh and Wong (2001)
		Tunnel and station construction at Race Course Road and Clarke Quay during the construction of North East Mass Rapid Transit Line	Wen (2005)
		Marina Coastal Expressway	LTA (2012); Shin et al. (2012)
		Marina Bay Financial Centre	Chen et al. (2011)
	Preventing basal heave failure	Geylang River canal	Ganeshan and Yng (2009)
	Controlling settlements	Tunnelling beneath Robinson Road	
Deep soil mixing	Foundation	Condominium at Sentosa Cove	Raju (2009)
	Excavation	Marina Bay Financial Center Project	Chen et al. (2011)
		Marina Coastal Expressway	Chun (2011); LTA (2012)
	Retaining structure	Waterway, Punggol	Soh et al. (2013)
	Deep excavation	Improved Berm for Multi-level train depot, Tanah Merah	Lai et al. (2016)
	Shaft construction/ Tunnel break-in/ break-out	Thomson East-Coast Line	
		Bedok South Station	

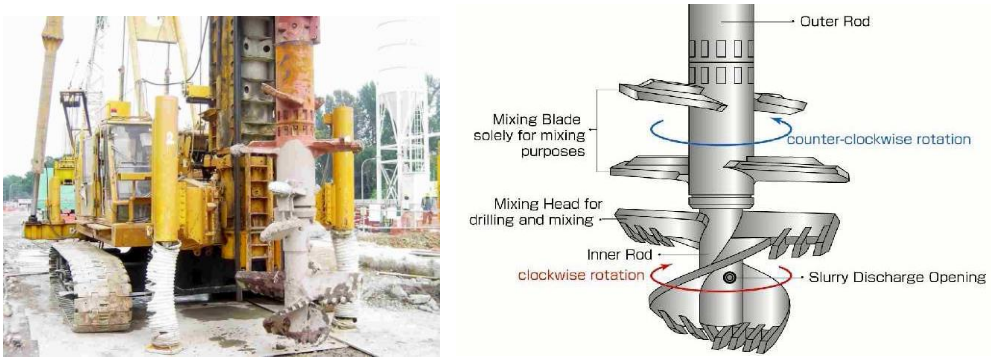
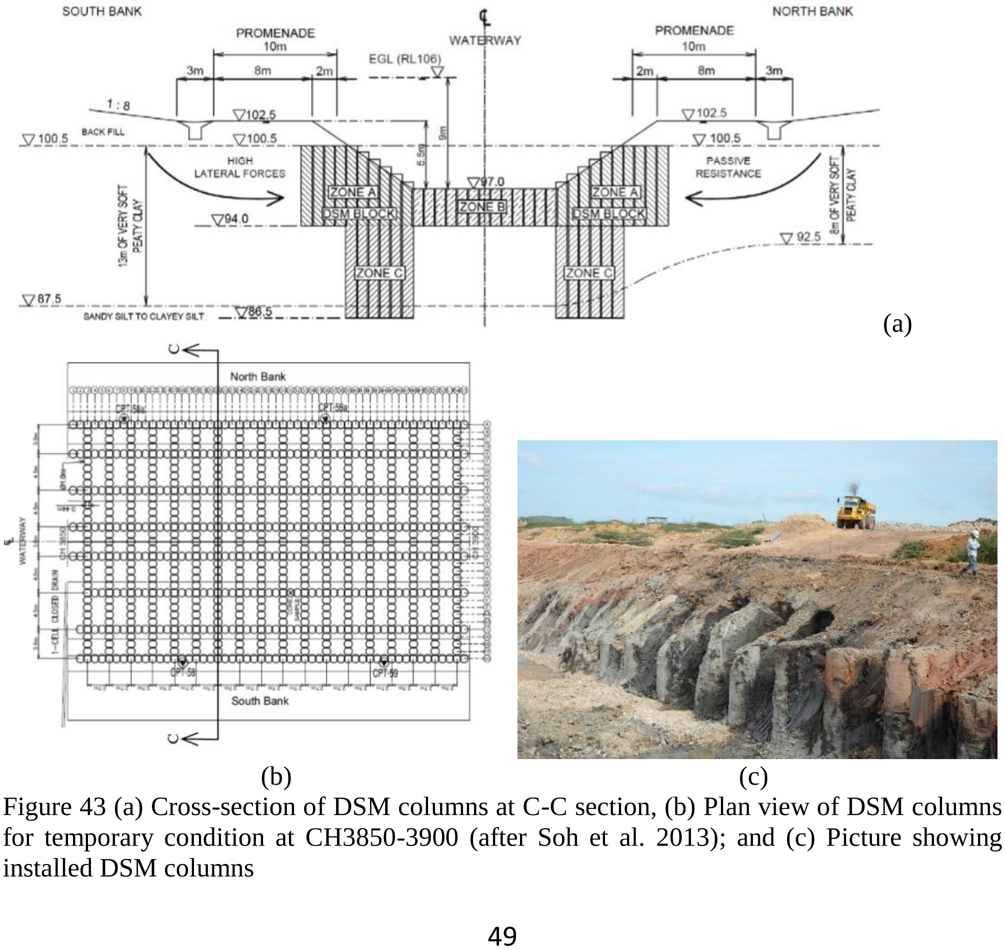


Figure 42. Jet mechanical mixing (JMM) machine and the drilling rod and mixing blades (after Osborne and Ng 2008)

5.4 Case study - DSM columns as retaining structures for waterway in Punggol

As part of the new water front town in Punggol, Singapore, a 4.2 km long waterway had to be constructed to connect two existing river reservoirs. The waterway was about 4 m deep, with a varying width of 24 to 80 m. The subsoil profile generally consists 5–7 m backfilled material (silty clay or loose clayey sand) overlying varying thickness of soft marine clay followed by typical Singapore Old Alluvium with varying degree of weathering. The maximum thickness of soft marine clay varied from 8 m to with strength (q_u) of less than 0.5 MPa. However, a 6 m layer of very soft marine clay with q_u of less than 0.25 MPa was found near the South Bank of the waterway. For area with very soft clay layer, CSM mixing columns of 1 m diameter with 0.44 m effective overlapping width were proposed in grid pattern as a retaining structure (Fig. 43a and 43c). The proposed configuration was found to utilize the advantage of high compressive strength and capable of isolating the development of tensile stresses. The average UCS of deep mixing columns was 2.93 MPa, which was well above the design UCS 800 kPa. The installed inclinometers were installed to monitor the deflections and only maximum 6 mm of deflection was registered, confirming satisfactory performance of grid arrangement. In addition, no sign of distress at the slopes and DSM columns were observed during the slope excavation for the concrete drain construction.



(a)

(b) (c)

Figure 43 (a) Cross-section of DSM columns at C-C section, (b) Plan view of DSM columns for temporary condition at CH3850-3900 (after Soh et al. 2013); and (c) Picture showing installed DSM columns

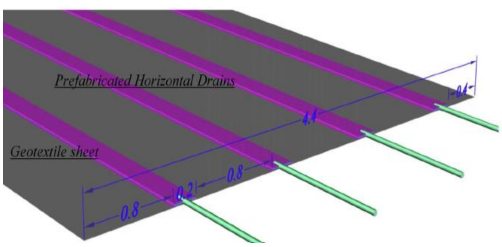
6 NEW DEVELOPMENTS

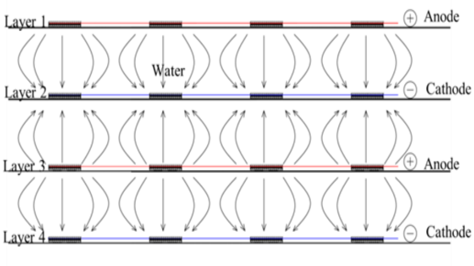
Ground improvement is a diversified, fast growing subject. Its state-of-the-practice is evolving all the time. Ground improvement is a practical driven discipline. It is not the method but the end result that matters. This poses challenges, but also gives opportunities for innovation. A review will not be complete without looking ahead of emerging technologies that could influence the ground improvement technologies in the future. A few of such methods are briefly introduced as follows.

6.1 Use of horizontal drains

PVDs have been widely used for the improvement of soft clay as discussed in Section 2. However, for land reclamation projects or projects involving placement of soft fill materials, the use of PVDs poses several problems: 1) PVDs can only be installed after all the fill has been placed and yet the treatment of soft soil using PVDs takes a long time. Thus, this method is too time consuming; 2) the top of the fills is too soft for PVDs to be installed. Special methods have to be adopted to form a working platform first; 3) For the improvement of the soft soil fill from the ground elevation, a relatively thick layer of fill needs to be topped up to compensate for the large settlement. The new fill will induce new consolidation and further settlement. Thus, it takes extra time for consolidation. It also requires a fairly accurate estimation of the settlement caused by fills placed at different times and yet the settlement of soft sill is large and difficult to predict. This become a difficulty for design.

One solution to this problem is to use horizontal drains. A patented new product, the so-called Horizontal Drainages enhanced Geotextile sheet (HDeG) has been developed by Chu and Guo (2015) as shown in Fig. 44a. Using this product, horizontal drains can be placed from a barge with the positions or spacing of the drains more or less maintained in-situ as designed. If there is a need to accelerate the consolidation process even further, electrolytes as either anode or cathode can be embedded into the HDeGs to create electro-osmosis effect that can also be incorporated into the horizontal drains as shown in Figure 44b.





(a) (b)

Figure 44 Horizontal Drainages enhanced Geotextile sheet (HDeG) (a) A typical design; and (b) HDeG arrangement and prevision to use electro-osmosis (After Chu and Guo, 2015)

There are a number of advantages in the use of horizontal drains with vacuum preloading: (1) Consolidation can take place as soon as the first layer of horizontal drains is covered by a clay layer as shown in Fig. 45; (2) The horizontal drains accelerate the sedimentation process of the clay mud layer and thus shorten the construction time; (3) The strength of the clay can be increased before the next layer of clay is placed; (4) With the use of horizontal drains, all the fills placed on top becomes the fill surcharge as well. So the fill surcharge load increases with the height of the fills; (5) The settlement can be predicted more reliably as the consolidation parameters can be back calculated from the consolidation of the clay fills placed earlier; and (6) The method is productive. The ground improvement works can be completed soon after the last layer of fill is placed. See Chu (2016) for more discussion.

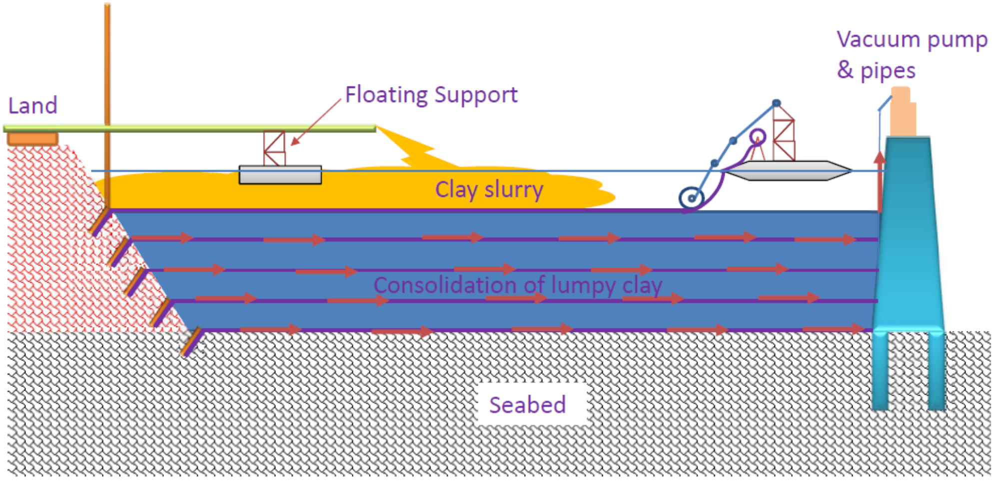


Figure 45 Schematic illustration of sequence for the placement of HDeG and clay fills.

6.2 Use of suction caissons as an alternative for offshore ground improvement

When large scale prefabricated concrete caisson walls are used for the construction of seawalls of ports, the soft seabed soil has to be treated or replaced to provide sufficient bearing capacity and control the settlement of the caissons. However, it is difficult, time consuming and expensive to carry out offshore ground improvement. An alternative is to use suction caissons (shown as the lower red cylinders in Fig. 46a) to penetrate into soft seabed to act as bearing layers for the caisson walls (shown as upper grey cylinders as shown in Fig. 46a). These upper and lower cylinders can be precast in the same way as for prefabricated concrete caissons and assembled together to form one unit at a casting yard before towed into the site for installation as shown in Fig. 46b. Once the unit is position, a suction is applied to the inner of the lower cylinders to suck the cylinders into the seabed soil. Suction caissons have been used as foundations for many bridge piers or offshore wind turbines projects. One project of using suction caissons for the construction of a breakwater offshore has been presented by Chu et al. (2015). Further discussion on the design and application of suction caisson can be found in Guo and Chu (2013), Guo et al. (2016).

(a) (b)

Figure 46 (a) Cylindrical concrete sea walls supported by suction caissons (after Chu et al. 2012c); and (b) Towing of suction caissons and sea wall unit to the installation site

6.3 Biogrouting

Cement or chemicals have often been used as binders for ground improvement. However, the use of cement or chemicals for ground improvement is not sustainable in the long run as cement or chemicals require a considerable amount of natural resource (for example limestone) and energy to produce. The use of cement or chemicals for ground improvement is also expensive. The cost of a ground improvement method itself is an important factor in deciding whether or not ground improvement works should be carried out to reduce the risks as discussed by Chu (2016). There is an urgent need to develop new ground improvement methods or new materials that can be used to reduce substantially the cost of ground improvement.

Using the latest microbial biotechnology, a new type of construction material, biocement, has been developed as an alternative to cement or chemicals (Whiffin, 2004; Ivanov and Chu, 2008; Chu et al. 2012). Biocement is made of naturally occurring microorganisms at ambient temperature and thus requires much less energy to produce. The main components of biocement can be made using wastewater sludge or other waste byproducts. The application of microbial biotechnology to ground improvement will also simplify some of the existing construction processes. For example, the biocement can be in either solid or liquid form. In liquid form, the biogrout has much lower viscosity and can flow like water. Thus, the delivery of biocement into soil is much easier compared to that of cement or chemicals. Furthermore, when cement is used, one usually has to wait for weeks for the full strength to be developed, whereas when biocement is used, the reaction time can be reduced if required.

A number of methods for the use of biotechnologies in ground improvement have been developed in recent years. These include (1) liquefaction mitigation method using biogas desaturation (He et al. 2013); (2) construction of temporary water storage pond in sandy soil using the biocrust method (Chu et al. 2012, 2013); (3) biogrouting for prevention of levee failure due to overtopping and erosion (Chu et al. 2015); (4) biogrouting of soil and rock joints; and (5) bio- calcification for creation of artificial coral reefs. Some of these methods are ready to be applied in practice although field testing is still required to develop the construction procedures. Nevertheless, it will be only a matter of time before some of those methods become reliable technologies. For sand and small gravels, both the shear strength and permeability properties can be greatly improved by biogrouting. For sand, the unconfined compressive strength achieved can be higher than 10 MPa and permeability be lower than 10^{-8} m/s. The stone block formed by biogrouting of a stone and sand mixture is shown in Fig. 47a. The impervious biocrust layer (with permeability of lower than 10^{-9} m/s) formed on top of a sand layer is shown in Fig. 47b. The testing for biogrouting of a rock joint is shown in Fig. 47c. The flow rate through the joint was reduced by more than 1000 times after biogrouting.

(a) (b) (c)

Figure 47 (a) Stone block formed by biogrouting of rock and sand mixture; (b) Impervious biocrust formed on top of sand (after Chu et al. 2012a); and (c) Biogrouting of a rock joint.

7 CONCLUDING REMARKS

In this paper, some ground improvement methods that are commonly adopted in Singapore are summarized. Some case histories in Singapore are also presented briefly to illustrate the applications. Some emerging technologies that have the potential to be used for future ground improvement works in Singapore are also introduced. In conclusion, we would like to quote Mitchell and Jardine (2002) "It is the nature of many ground treatment techniques that their capability is continually being extended, overcoming what were previously seen as limitations. Moreover, different techniques can be combined to cope with a greater range of situations than one method on its own."

8 ACKNOWLEDGEMENT

The authors acknowledge gratefully the assistance of Ms Akanksha Tyagi from Keller Foundations (S E Asia) Pte Ltd, Singapore, for collecting information for this paper.

9 REFERENCES

Abuel-Naga, H.M. and Bouazza, A. 2009. On the equivalent diameter of prefabricated vertical drain: numerical study. *Geotextiles and Geomembranes* 27(3), 227-231.

Akagi, T. 1994. Hydraulic application of geosynthetics to filtration and drainage problems – with special reference to prefabricated band-shaped drains. *Proc. 5th Int. Conf. on Geotextiles, Geomembranes and Related Products, Special Lecture*, 4, Singapore, 97-119.

Arulanantham, S. and Yogarajah, I. 2016. Ground Improvement using wet speed mixing method (WSM), *Proc 19th South East Geotechnical Conference & 2nd AGSSEA Conference*, Kuala Lumpur, 489-493.

Arulrajah, A., Bo, M.W. and Chu, J. 2009. Instrumentation at the Changi land reclamation project, Singapore. *Geotechnical Engineering*, ICE, 162(1), 33-40.

Asaoka, A. 1978. Observational procedure of settlement prediction. *Soils and Foundations* 18(4), 87-101.

Australian Standards, AS8700 2011: *Execution of prefabricated vertical drains*, Standards Australia.

Balasubraniam, A.S., Bergado, D.I., Long, P.V. and Thayalan 1995. Experiences with sand drains and prefabricated vertical drains in ground improvement of soft clays, *Seminar on Engineering for Coastal Development*, Singapore.

Barron, R.A. 1948. Consolidation of fine-grained soils by drain wells. *Trans ASCE*, 113, Paper 2346, pp. 718-54.

Basu, D. and Prezzi, M. 2007. Effect of the Smear and Transition Zones around Prefabricated Vertical Drains Installed in a Triangular Pattern on the Rate of Soil Consolidation. *International Journal of Geomechanics*, ASCE, 7(1), 34-43.

Bergado, D.T., Asakami, H., Alfaro, M.C. and Balasubramaniam, A.S. 1991. Smear effects of vertical drains on soft Bangkok clay. *J. Geotech. Eng.*, ASCE, 117(10), 1509-1530.

Bergado, D.T., Alfaro, M.C., and Balasubramaniam, A.S. 1993. Improvement of soft Bangkok clay using vertical drains, *Geotextiles and Geomembranes* 12(7), 615-663.

Bergado, D.T., Balasubramaniam, A.S., Fannin, R.J. and Holtz, R.D. 2002. Prefabricated vertical drains (PVDs) in soft Bangkok clay: a case study of the New Bangkok International Airport project, *Canadian Geotechnical Journal*, 39, 304–315.

Bergado, D.T., Chai, J.C., Miura, N. & Balasubramaniam, A.S. 1998. PVD improvement of soft Bangkok clay with combined vacuum and reduced sand embankment preloading. *J. Geot. Eng., Southeast Asian Geot. Soc.*, *2*9(1): 95-122.

Bo, M.W., Chu, J., and Choa, V. 2005. Changi East Reclamation and Soil Improvement Project. In: Chapter 9, *Ground Improvement—Case Histories*. Indraratna, B., Chu, J. (Eds.), Elsevier, Amsterdam, 247–276

Bo, M.W., Chu, J., Low, B.K. and Choa, V. 2003 *Soil Improvement: Prefabricated Vertical Drain Technique*, Thomson Learning, Singapore.

BS EN 15237 2007. *Execution of Special Geotechnical Works — Vertical Drainage*. European Standard.

BS EN 12716 2001. *Execution of Special Geotechnical Work –Jet Grouting.* European Standard.
BS EN 12715. 2000. *Execution of special geotechnical work - Grouting*. European Standard.
BS EN 14679. 2005. *Execution of special geotechnical works - Deep mixing*. European Standard.
Burke, G.K., Cacoilo, D.M., & Chadwick, K.R. 2000. Super jet grouting: new technology for In situ soil improvement. *Transportation Research Record: Journal of the Transportation Research Board*, No. 1721: 45-53.
Carillo, N. 1942. Simple two - and three - dimensional cases in the theory of consolidation of soils. *Journal of Mathematics and Physics,* 21(1), 1-5.
Carroll, R.G. 1983. Geotextile filter criteria. *Transportation Research Record*, 916, 46-53.
Chai, J. C., Miura, N., and Bergado, D. T. 2008. Preloading clayey deposit by vacuum pressure with cap-drain: Analyses versus performance. *Geotext. Geomembranes*, 26(3): 220-230.
Chai, J.C. and Miura, N. 1999. Investigation of factors affecting vertical drain behavior. *Journal of Geotechnical Engineering*, ASCE, 125(3): 216-226
Chen, H., & Bao, X.C. 1983. Analysis of soil consolidation stress under the action of negative pressure. *Proceedings of the European Conference on Soil Mechanics and Foundation Engineering*, 2: 591-596.
Chen, J., Lee, F.H., and Ng, C.C. 2011. Statistical analysis for strength variation of deep mixing columns in Singapore. *Proc Geo-Frontiers 2011: Advances in geotechnical engineering* (eds J. Han and D. E. Alzamora), ASCE GSP No. 211, 576–584.
Choa, V. 1980. Geotechnical aspects of a hydraulic fill reclamation. *Proc. 6th Southeast Conf. on Soil Engr.*, Taipei. Volume 1, 469-484.
Choa, V., Bo, M.W., and Chu, J. 2001. Soil improvement works for Changi East reclamation project. *Ground Improvement,* 5(4), 141–153.
Choa, V., Vijiaratnam, A. Karunaratne, G.P., Ramaswamy, S.D. and Lee, S.L. 1979. Pilot Test for Soil Stabilization at Changi Airport, *Proceedings 7th European Regional Conference on Soil Mechanics and Foundation Engineering,* Brighton, 3, 29-36.
Choa, V., Bo, M.W., Arulrajah A. and Na Y.M. 1997. Overview of densification of granular soil by deep compaction methods. *Proc Int. Conf Ground Improvement Techniques*, Macau, 131– 140.
Chu, J. 2016. Innovations in soil improvement methods. S.L. Lee Lecture, *Proc 19th Southeast Asian Geotechnical Conf. & 2nd AGSSEA Conf*, 31 May – 3 June 2016, Subang Jaya, Malaysia.
Chu, J. and Guo, W. 2015. A geotechnical apparatus and method of usage, *SG Patent Application, Serial No. 10201509039Y* on 3 November.
Chu, J. and Raju, R.V. 2012. Prefabricated vertical drain. *Chapter 4 in Ground Improvement*, 3rd Edition, Eds. K. Kirsch and A. Bell, CRC Press, 87-167.
Chu, J. and Yan, S.W. 2005a. Estimation of degree of consolidation for vacuum preloading projects, *International Journal of Geomechanics*, ASCE, 5(2), 158-165.
Chu, J. and Yan, S.W. 2005b. Application of the vacuum preloading method in land reclamation and soil improvement projects. Chapter 3, In *Ground Improvement – Case Histories*, Eds. B. Indraratna and J. Chu, Elsevier, 91-118.
Chu, J., Bo, M.W. and Arulrajah, A. 2009a. Soil improvement works for an offshore land reclamation. *Geotechnical Engineering, ICE,* 162(1), 21–32.
Chu, J., Bo, M.W. and Arulrajah, A. 2009b. Reclamation of a slurry pond in Singapore, *Geotechnical Engineering, ICE*, 162(1), 13–20.
Chu, J., Varaksin, S., Klotz, U. and Mengé, P. 2009c. Construction processes. *State-of-the-Art-Report, Proc. 17th Int Conf on Soil Mechanics and Geotechnical Engineering*, Alexandria, Egypt, 5-9 Oct., 4, 3006-3135.

Chu, J., Bo, M.W. and Choa, V. 2004. Practical consideration for using vertical drains in soil improvement projects, *Geotextiles and Geomembranes,* 22, 101-117.
Chu, J., Bo, M.W. and Choa, V. 2006. Improvement of ultra-soft soil using prefabricated vertical drains, *Geotextiles and Geomembranes*, 24(6), 339-348.
Chu, J., Bo, M.W., Chang, M.F., and Choa, V. 2002. Consolidation and permeability properties of Singapore marine clay, *J. Geotechnical and Geoenvironmental Engineering*, ASCE, 128(9), 724-732.
Chu, J., Indraratna, B., Yan, S.W. and Rujikiatkamjorn C. 2014. Overview of preloading methods for soil improvement. *Ground Improvement, ICE*, 167(3), 173-185.
Chu, J., Stabnikov, V. and Ivanov, V. 2012a. Microbially induced calcium carbonate precipitation on surface or in the bulk of soil, *Geomicrobiology Journal,* 29, 544–549.
Chu, J., Yan, S.W., and Lam, K.P. 2012b. Methods for improvement of clay slurry or sewage sludge, *Ground Improvement, ICE*, 165(4), 187-199.
Chu, J., Yan, S.W., Li, W. 2012c. Innovative methods for dike construction – An overview. *Geotextiles and Geomembranes,* 30(2), 35-42.
Chu, J., Ivanov, V., Stabnikov, V. and Li, B. 2013. Microbial method for construction of aquaculture pond in sand. *Geotechnique*, 63(10), 871-875.
Chu, J., Ivanov, V., He, J., Maeimi, M., Wu, S, 2015. Use of biogeotechnologies for soil improvement, *Chapter 27 in Ground Improvement Case Histories: Chemical, Electrokinetic, Thermal, and Bioengineering Methods*, Eds. Indraratna et al, 571-590.
Chu, J., Yan, S.W. and Yang, H. 2000. Soil improvement by the vacuum preloading method for an oil storage station. *Geotechnique*, 50(6), 625-632.
Chu, J., Yan, S. W., and Indraratna, B. 2008. Vacuum preloading techniques – recent developments and applications. *Geosustainability and Geohazard Mitigation, GeoCongress 200*8, ASCE Geotechnical Special Publication No. 178, 586-595
Chu, J., Yan, S.W., and Guo, W. 2016. Vacuum preloading methods: an update, *Geotechnical Engineering Journal of the SEAGS & AGSSEA,* 47(3), 62-69.
Chun, Y.C. 2011. DSM application for deep excavation in Singapore. *Journal of the Korea-Academia-Industrial Cooperation Society*. 12(5), 2425-2433.
Cognon, J.M. 1991. Vaccum consolidation. *Révue Française Géotechnique*, 57, 37-47.
Daramalinggam, J., Leong K.W. and Partzsch, S. 2009. Comprehensive site investigation and construction monitoring for tank farms on improved ground. *Ground Improvement Technologies and Case Histories*. Edited by C. F. Leung, J. Chu and R. F. Shen. Research Publishing Services. 647-654.
Essler, R., & Kitazume, M. 2008. *Application of Ground Improvement: Deep Mixing*. TC17 website: www.bbri.be/go/tc17
Gaba, A.R. 1990. Jet Grouting at Newton Station, Singapore. *Proc 10th Southeast Asian Geotechnical Conference*, Taipei. 77-79.
Ganeshan, V. and Yng, J.Y. 2009. Jet grouting and its applications. *Ground Improvement Technologies and Case Histories*, edited by C. F. Leung, J. Chu and R. F. Shen. Research Publishing Services, 209-220.
Guo, W. and Chu, J. 2013. Suction caisson installation in shallow water: model tests and prediction. *Proc. 18th Int. Conf on Soil Mech. and Geot. Eng.*, Paris, 2-6 Sept, 1999-2002.
Guo, W., Chu, J. and Kou, H.L. 2016. Model tests of soil heave plug formation in suction caisson, *Geotechnical Engineering, ICE*, 169(2), 214-223
Hansbo, S. 1979 Consolidation of clay by band-shaped prefabricated drains, *Ground Engineering*, 12(5), 16-25.
Hansbo, S. 1981 Consolidation of fine-grained soils by prefabricated drains, *Proc. 10th International Conference Soil Mechanics and Foundation Engineering.*, Stockholm, Vol. 3, pp. 677-682.

Hansbo, S. 2005. Experience of consolidation process from test areas with and without vertical drains. In B. Indraratna & J. Chu (Eds.), *Chapter 1, Ground Improvement - Case Histories*, Elsevier, pp. 3-50.

Hashimoto, T. and Liu, Y. 2011. New technologies for underground construction in soft ground of urban area, *Proc China's 5th Int Symp on Tunneling.*

He, J., Chu, J., and Ivanov, V. 2013. Mitigation of liquefaction of saturat-ed sand using biogas. *Geotechnique*, 63(4), pp 267-275.

He, Z.W., Leong, K.W. and Selvaraju, S. 2015. Steel tanks on deep vibro techniques improved ground. *Advances in Soft Ground Engineering*. Edited by C. F. Leung, T. Ku and S. C. Chian, Research Publishing, Singapore. 387-397.

He, Z.W., Lin, E., Leong, K.W. and Raju, V.R. 2016. Design and Performance of ground improvement for a ship hull workshop in Singapore. *19th Southeast Asian Geotechnical Conference & 2nd AGSSEA Conference*, Kuala Lumpur. 469-473.

Hird, C.C. and Moseley, V.J. 2000 Model study of seepage in smear zones around vertical drains in layered soil. *Geotechnique* 50(1), 89-97.

Holtz, R.D. 1987 Preloading with prefabricated vertical strip drains, *Geotextiles and Geomembranes*, 6(1–3), 109–131.

Holtz, R.D. 1975. Preloading by vacuum: current prospects. *Transportation Research Record*, 48(5), 26-79.

Holtz, R.D. and Kovacs, W.D. 1981 *An Introduction to Geotechnical Engineering*, Prentice-Hall.

Holtz, R.D., Jamiolkowski, M., Lancellotta, R. and Pedroni, R. (1991) *Prefabricated Vertical Drains: Design & Performance*, CIRIA Ground Engineering Report, Butterworth-Heinemann Ltd., London.

Indraratna, B. and Redana, I.W. 1998. Laboratory determination of smear zone due to vertical drain installation, *J. Geotech. Eng.*, ASCE, 124(2), 180-184.

Indraratna, B., Balasubramaniam, A. S. and Ratnayake, P. 1994. Performance of embankment stabilized with vertical drains on soft clay. *J. Geotech. Eng*., ASCE, 120(2), 257-273.

Indraratna, B., Rujikiatkamjorn, C., Ameratunga, J., and Boyle, P. 2011. Performance and Prediction of Vacuum Combined Surcharge Consolidation at Port of Brisbane. *J. of Geot. & Geoenv Eng.*, ASCE, 137 (11), 1009-1018.

JTJ/T256-96 1996. *Code of Practice for Installation of Prefabricated Drains*. Ministry of Communications, China.

JTJ/T257-96 1996. *Quality Inspection Standard for Prefabricated Drains*. Ministry of Communications, China.

Kirsch, K. and Kirsch, F. 2017. Ground Improvement by Deep Vibratory Methods. Second Edition. CRC Press, Taylor & Francis Group, New York.

Kitazume, M. (2007) Design, execution and quality control of ground improvement in Land reclamation, *Proc. of the 13th Asian Regional Conference on Soil Mechanics and Foundation Engineering*, Keynote Lecture.

Kjellman W. 1952, Consolidation of clayey soils by atmospheric pressure. Proceedings of a Conference on Soil Stabilisation, MIT, Boston: 258–263.

Kolff, A.H.N., Spierenburg, S.E.J. and Mathijssen, F.A.J.M. (2004) BeauDrain: a new consolidation system based on the old concept of vacuum consolidation, *Proc. 5th International Conference on Ground Improvement Techniques,* Kuala Lumpur, Malaysia.

Kremer, R.R.H.J., Oostven, J.P., Van Weele, A.F., Dejager, W.F.J., and Meyvogel, I.J. (1983) The quality of vertical drainage, *Proc. 8th European Conf. on Soil Mechanics and Foundation Engineering*, Vol. 2, Helsinki, pp. 721-726.

Lai, W.T., Ye, Q.Y. and Chepurthy, V. 2016. Geotechnical Design Aspects for an Underground Multilevel Train Depot, *Underground Singapore 2016*, Tunnelling and Underground Construction Society (Singapore).

Land Transport Authority (LTA). 2012. *Marina Coastal Expressway Contract 482 Fact Sheet*.

Land Transport Authority (LTA). 2013. Annual Report 2012/2013. Retrieved on 27th September 2017 from https://www.lta.gov.sg

Larsson, S.M. 2005. State of practice report - Execution, monitoring and quality control, *International Conference on Deep Mixing*: 732-785.

Leong, K.W. and Raju, V.R. 2007. Use of Vibro Replacement method for the stabilization of an offshore slope on soft clay. *Proc the sixteenth Southeast Asian Geotechnical Conference, Malaysia*, South Asian Geotechnical Society, pp. 547-550.

Leong, K.W., He, Z.W. and Haneena, M. 2013. Vibro compaction for Chemical Plants in Jurong Island. *Advances in Geotechnical Infrastructure*, Edited by C. F. Leung, S. H. Goh and R. F. Shen. Research Publishing. 171-175.

Leung, C.F. 2013. Ground improvement techniques for land reclamation-Singapore experience. *Proc Int Conf Ground Improvement and Ground Control, Transport Infrastructure Development and Natural Hazards Mitigation*. Edited by B. Indraratna, C. Rujikiatkamjorn and J. Vinod. Published by Research Publishing. 1, 193-208.

Lo, K.W., Ooi, P.L. and. Lee, S.-L 1990a. Unified approach to ground improvement by heavy tamping. *J. Geotech. Engng.*, ASCE, 116(3), 514–527.

Lo, K. W., Ooi, P. L. and. Lee, S.-L 1990b. Dynamic replacement and mixing of organic soils with sand charges. *J. Geotech. Engng*., ASCE, 116(10), 1463–1482

Lukas R.G. 1995. *Dynamic Compaction*, US Department of Transportation, FHWA-SA-95-037.

Madhav, M.R., Park, Y.M. and Miura, N. 1993. Modelling and study of smear zones around band shaped drains, *Soils and Foundations*, 33(4), 135-147.

Mayne, P.W., Jones, J.S. and Dumas, J.C. 1984. Ground Response to Dynamic Compaction. *Journal of Geotechnical Engineering*, ASCE. 110(6), 757-774.

Mitchell, J.M., & Jardine, F.M. 2002. *A Guide to Ground Treatment*. CIRIA.

Narendranathan, N. and Lee, E.C. 2015. Dynamic Compaction and Dynamic Consolidation of Soils. *Ground Improvement Case Histories-Compaction, Grouting, and Geosynthetics*, Elsevier. pp. 365-398.

Ong, K., Yee, K. & Wong, L.T. 2007. Dynamic compaction and dynamic replacement for large oil tank, *Proc. 16th Southeast Asian Geotechnical Conference*, 8-11May, Kuala Lumpur: 591-595.

Osborne, N.H., & Ng, C.C. 2008. Strut Omission by Observational Approach for Deep Excavation in Singapore using Hybrid Ground treatment, *Proc Int Conf on Deep Excavations*. Tunnelling and Underground Construction Society of Singapore.

Page, R.J., Ong, J.C.W., and Osborne, N. 2006. Jet grouting for excavations in soft clay-Design and Construction issues. *Proc Int Conf on Deep Excavations*, Tunnelling and Underground Construction Society of Singapore.

Poh, T.Y., and Wong, I.H. 2001. A field trial of jet-grouting in marine clay. *Canadian Geotechnical Journal*. 38(2), 338-348.

Porbaha, A., Tanaka, H., and Kobayashi, M. 1998. State of the art in deep mixing technology, Part II: Application areas, *Ground Improvement*, 2(3), 125-139.

Port of Singapore Authority (PSA). 2017. *Terminals*. Retrieved on 27th September 2017 from https://www.singaporepsa.com/our-business/terminals

Raju, V.R. 2009. Ground Improvement- Principles and Applications in Asia. *Ground Improvement Technologies and Case Histories*, Singapore. Edited by C. F. Leung, J. Chu and R. F. Shen. Research Publishing. 43-65.

Raju, V.R. and Sondermann, W. 2005. Ground Improvement using Deep Vibro Techniques. *Ground Improvement- Case Histories*. Edited by B. Indraratna and J. Chu. Elsevier, UK. 601-638.

Raju, V.R., Krishna, R.H., and Wegner, R. 2004. Ground improvement using Vibro Replacement in Asia 1994 to 2004- A 10year review. *5th Int. Conf. Ground Improvement Techniques*, 22 – 23 March, Kuala Lumpur, Malaysia.

Raju, V.R., Yee, Y.W., and Daramalinggam, J. 2016. Vibro Stone Columns: Proper Construction Practices. *Reclamation: Challenges and Beyond, The HKIE Geotechnical Division 36th Annual Semina*r. 183-192.

Ramaswamy, S.D., Aziz, M.A., Subrahamanyam, R.V., Abdul Khader, M.H., and Lee, S.-L. 1979. Treatment of peaty clay by high energy impact. *J. Geotech. Engng. Div*., ASCE. 105(8), 957–967.

Saowapakbiboon, J., Bergado, D.T., Chai, J.C., Kovittayanon, N., and de Zwart, T.P. 2008. Vacuum-PVD combination with embankment loading consolidation in soft Bangkok Clay: A Case study of Suvarnabhumi Airport project, *Proc. 4th Asian Regional Geosynthetics Conference*, Shanghai, China, pp. 440-449.

Sathananthan, I. and Indraratna, B. 2006. Laboratory evaluation of smear zone and correlation between permeability and moisture content. *J. Geotech. Geoenviron. Eng*., 132(7): 1090-0241.

Seah, T.H. 2006. Design and construction of ground improvement works at Suvarnabhumi Airport. *Geotechnical Engineering Journal of Southeast Asian Geotechnical Society*, 37, 171-188.

Shin, E.C., Park, Y.J. and Park, J.J. 2012. Lateral displacement of temporary retaining wall as jet grouting pile installation in Singapore. *3rd Int Conf on New Developments in Soil Mechanics and Geot Eng*. North Cyprus. 87-93.

Shinsaka, T., Yamazaki, J., Nakanishi, Y. and Komiya, K. 2017. Development of the multifan-shaped jet grouting method of ground improvement. *Proc Greouting 2017*, GSP289, ASCE.

Singapore Land Authority (SLA). 2017. *Total land area of Singapore. Government of Singapore*. Retrieved on 8th September 2017 from https://data.gov.sg/dataset/total-land-area-of-singapore

Soh, K.K., Leong, W.K., Yeo, L., Chew, S.H., Leong, K.W. and He, Z.W. 2013. Deep soil mixing columns as retaining structure. *Advances in Geotechnical Infrastructure.* Edited by C. F. Leung, T. Ku and S. C. Chian, Research Publishing, Singapore. 157-170.

Sridharan, A. and Sreepada Rao, A. 1981. Rectangular hyperbola fitting method for one-dimensional consolidation, *Geotechnical Testing Journal*, 4(4), 161-168.

Sugawara, S., Shigenawa, S., Gotoh, H., and Hosoi, T. 1996. Large-scale jet grouting for pre-struting in soft clay. *Proc 2nd Int Conf on Ground Improvement Geosystems: Grouting and Deep Mixing*. A.A. Balkema, Rotterdam. 353-356.

Tan, T.S., Goh, T.L., Yong, and K.Y., 2002. Properties of Singapore marine clays improved by cement mixing. *Geotechnical testing Journal*, 25(4), 1-12.

Tan, T.S., Inoue, T. & Lee, S.L. 1991. Hyperbolic method for consolidation analysis. *J. Geotech. Engng*. ASCE, 117(1), 1723-1737.

Tan, S.A. 1993. Ultimate settlement by hyperbolic plots for clays with vertical drains. *J. Geotech. Engng.* ASCE, 119(5), 950-956.

Tang, M. and Shang J.Q. 2000. Vacuum Preloading Consolidation of Yaoqiang Airport Runway. *Geotechnique*. 50(6), 613-623.

Topolnicki, M., 2004. Insitu Soil Mixing. *Ground improvement*, M.P.Moseley and K. Kirsch, eds., Vol 2, Taylor & Francis, London, 331-428.

Vanmarcke, E.H. 1977. Probabilistic modelling of soil profiles, *Journal of the Geotechnical Engineering Division*, 103(11), 1227-1246

Varaksin, S. & Yee, K. 2007. Challenges in ground improvement techniques for extreme conditions: Concept and Performance, *Proc.16th Southeast Asian Geotechnical Confeence*, 8-11 May: Kuala Lumpur, 101-115.

Wei, J. 1997. Ground improvement techniques- the HDB way. *Proc Intl Conf on Ground Improvement Techniques*, Macau, 609-619.

Wei, J. and Khoo, T.K. 1992. Reclamation at Marina Bay and Tanjung Rhu, *ECD Proc Seminar on Engineering for Coastal Development*, Nov 17-28, Singapore, 39–45.

Wei, J., Chua, K.C., and Teo, Y.P. 1993. Treatment of Marine Clay Pits on Reclaimed Land at Pasir Ris using Dynamic Replacement Method. *Proc. Eleventh Southeast Asian Geot. Conf.*, 4-8 May, Singapore. 429-433.

Welsh, J.P. and Burke, G.K. 2000. Advances in grouting technology, *Proc GeoEng 2000*. Melbourne.

Wen, D. 2005. Use of Jet Grouting in Deep Excavations. Ground Improvement using Deep Vibro Techniques. *Ground Improvement- Case Histories*. Edited by B. Indraratna and J. Chu. Elsevier, UK. 3, 357-370.

Yan, H.S., & Cao, D.Z. 2005. Application of low-level vacuum preloading technique in offshore projects. *Ocean and River Hydraulics* (3): 41-43 (in Chinese).

Yan, S.W. and Chu, J. 2005. Soil improvement for a storage year using the combined vacuum and fill preloading method. *Canadian Geotechnical Journal*, 42(4), 2094-1104.

Yan, S.W., and Chu, J. 2003 Soil Improvement for a road using the vacuum preloading method, *Ground Improvement*, 7(4), 165-172.

Yan, S.W., Chu, J., Fan, Q.J., & Yan, Y. 2009. Building a breakwater with prefabricated caissons on soft clay. *Geotechnical Engineering*, *ICE*, 162(1), 3-12.

Yoshikuni, H. and Nakanodo, H. 1974. Consolidation of Fine-Grained Soils by Drain Wells with Finite Permeability. *Soil Mech. and Found. Eng*. 14(2), 35-46.

Zeng, G.X. and Xie, K.H. 1989. New development of the vertical drain theories. *Proce. of the 12th Inter. Conf on Soil Mech. and Foundation Eng*. Rotterdam 2: 1435-1438.

Zheng, Y.R., Chu, J., Lu, X., & Feng, Y.X. 2004. Improvement of soft ground by dynamic compaction. *Geotechnical Engineering*, Journal of Southeast Asian Geotechnical Society, 35(1): 39-46.