Innovative Soft Soil Improvement Method through Intelligent Use of Vacuum De-Watering and Dynamic Compaction Techniques

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ABSTRACT: Recently, an innovative soft soil improvement method was advanced in China by integrating and modifying vacuum consolidation and dynamic compaction ground improvement techniques in an intelligent and controlled manner. This innovative soft soil improvement method, often referred to as "High Vacuum Densification Method (HVDM)", has been successfully used in China and Asia for numerous large-scale soft soil improvement projects, from which enormous time and cost savings have been achieved. In this presentation, the working principles of the HVDM are described, followed by a summary of two case studies. Results of numerical simulation using FLAC3D computer program, in which dynamic compaction is modelled as a three-dimensional, coupled hydro-mechanical model, are presented to highlight the mechanisms of positive pore pressure generation due to dynamic compaction. Both field studies and numerical simulation results support the mechanisms of HVDM in that dynamic compaction induced positive pore water pressure together with vacuum generated negative pore water pressure have added effects in rapidly expelling water out of the soil, thus increasing density (reducing void ratio) and improving undrained shear strength of soft, fine-grained soils in a relatively short duration.

KEYWORDS: HVDM, Soil improvement, Vacuum consolidation, FLAC3D

1. INTRODUCTION

In-situ improvement of soft cohesive soils is one of the main challenges facing geotechnical engineers and contractors alike. In countries such as China, India, and other emerging countries in Asia where the population is large and infrastructure development is proceeding at a heightened pace, the need for a fast, economical in-situ improvement for soft cohesive soils in a large-scale is clearly evident. The traditional methods of soft cohesive soil treatment include the use of the following techniques: (a) prefabricated vertical drains (PVDs) and fill preloading, (b) vacuum consolidation together with PVDs, (c) stone columns, (d) thermal treatment, (e) chemical mixing, (f) electro-osmosis, and (f) deep dynamic compaction. Despite the availability of various methods of in-situ improvements listed above, the method of incorporating PVDs with fill preloading appears to be the most widely used technique throughout the world, even though the vacuum consolidation method has gained some interest recently. In large-scale applications — such as land reclamation using dredged materials, port facility construction, economic zone development along coastal areas, petro-chemical plants near shorelines, steel mills, power plants, airport runways, and highways — the areas to be treated could be excessively large and the availability of usable earth for fill preloading could be scare. Therefore, there is a great interest in developing a more effective way of treating soft cohesive soils in a large area where preloading fill cannot be economically obtained.

The application of vacuum to facilitate consolidation in saturated fine grained soils has been used either alone by means of PVDs or in combination with the static surcharge load using fill materials (Kjellman 1952 and Holtz 1975). The effectiveness of vacuum consolidation with or without surcharge loading is highly dependent upon soil permeability and the efficiency of the vacuum system. The desired degree of soil improvement and the allowable time duration for completion can also play an important role in determining if vacuum consolidation can be a viable soil improvement method for the project. The use of deep dynamic compaction technique in saturated fine grained soils has not been widely accepted, even though Menard (1975) has demonstrated the validity of its working mechanisms.

Vacuum consolidation has been a subject of intensive research over the years. The publication on the subject is too numerous to list all of them. The design method contained in Elias, et al (2006) is commonly used in the United States. Bergado et al (1998) provide an alternative approach by using the finite element method. Indraratna et al (2005a, 2005b) contribute to the state-of-art knowledge on vacuum consolidation with the use of PVD. Interesting case studies were documented in Chu and Yang (2005a, 2005b) in which a combination of vacuum consolidation together with embankment fill preloading was used successfully. More recently, wide usage of vacuum consolidation can be seen in China, as exemplified by publications, such as Yan et al (2009) and Yan and Chu (2010), among others.

Due to rapid infrastructure development in China, an innovative soft cohesive soil treatment technique was developed in 2000 and had since been rapidly applied in China and other countries in Asia. The core of this innovative, in-situ, cohesive soil treatment method was termed as "High Vacuum Densification Method" (HVDM), and it was granted a series of international patents and registered in more than 25 countries. The success of HVDM was quite remarkable in a sense that the technique blends two well-known soil improvement methods, vacuum consolidation and deep dynamic compaction, into an intelligent yet efficient soft soil treatment method that can treat a large area within a relatively short time period.

In this paper, the working principles of this ground improvement method are described. The distinguishing features of this method and its advantages and limitations are elucidated. The mechanisms of undrained strength gain due to this method are also illustrated. Finally, a field case is presented at the end of this paper. It should also be noted that this method is limited to treating soft soils to depths up to 8 to 10 meters. For treating the soft soils with depth greater than 8 meters or for achieving much higher improved strength, the conventional surcharge loading with prefabricated vertical drains (PVD) is required.

2. ROOTS OF HVDM AND ITS CONSTRUCTION SEQUENCES

HVDM is a soft soil treatment method that is fast and combines vacuum drainage and deep dynamic compaction in designated cycles, so that soils at the project site can be improved through the effects of lowered water content and increased density. By using this method, soil strength and stiffness are improved, and the total and differential settlements after HVDM treatment are minimized.

The development of HVDM can be traced back to early 2000, when the inventor, Mr. Shi-Long Xu of Shanghai Geoharbour Group, began experimenting with the concept of high vacuum densification and applying it in a large scale to many well-known projects around Shanghai, such as Shanghai Pudong Airport Runway No. 2, Shanghai International Circuit, and the Shanghai port expansion. Mr. Xu later filed patent applications and received Patent Cooperation Treaty approval for several separate but related soft ground improvement technologies. Among the three main patents are patent no. ZL 01127046.2, involving the use of multiple cycles of high vacuum process and varied dynamic compaction efforts (or mechanical compaction) to reduce water content in soft soils; patent no. ZL 200410014257.9, involving the combined use of surcharge preloading or vacuum consolidation, followed by HVDM; patent no. ZL 200510134966.5, involving the use of HVDM followed by construction of stone columns or other types of composite foundations. After initial successful applications in the Shanghai area, HVDM was expanded into other areas in China and other countries in Asia, such as Vietnam, Malaysia, and Indonesia. Currently, HVDM has become a major method used in land reclamation projects along coastal areas in China, with over 9 million meters square of land treated in the last 7 to 8 years.

HVDM can be described as a fast ground improvement technology utilizing drainage, consolidation, and densification principles. HVDM is generally executed in a controlled manner based on feedback of on-site monitoring data collected for quality assurance and quality control (QA/QC) purposes. Figure 1 provides a schematic drawing of HVDM using vacuum consolidation and deep dynamic compaction. The HVDM consists of the following steps:

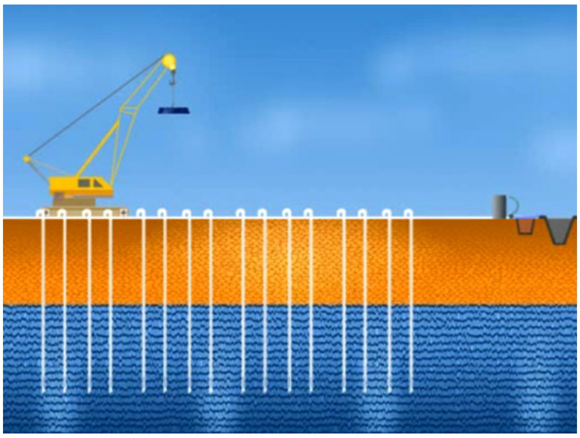


Figure 1 Schematic showing the HVDM method

Step 1: Conduct detailed geotechnical investigation at the project site. Evaluate and determine the soil profile at the site with detailed knowledge of the depth and thickness and distribution pattern of soft soils requiring treatment. Obtain important basic soil properties, including gradation curves, Atterberg limits, water content, hydraulic conductivity, compressibility, and coefficient of consolidation. Conduct in-situ tests, such as cone penetration testing (CPT) or soil test phosphorus (STP) to establish baseline values prior to commencing HVDM in the field. Understand and establish performance criteria of ground treatment. Perform preliminary design to provide plans for optimum spacing and depth of vacuum pipes, energy level of deep dynamic compaction (DC) and number of drops and grid spacing of tamper, time needed for vacuum consolidation between cycles of dynamic compaction, etc. However, it should be emphasized that the initial plans will generally need to be modified based on on-site monitoring data and the expected final performance criteria.

Step 2: Install vertical vacuum pipes and horizontal drainage pipes. The vertical vacuum pipes can be installed using several different methods, such as using a vibratory hammer and a mandrel or employing a hydraulic system to directly push vacuum pipes into ground. It is noted that vacuum pipes are steel pipes, typically 1 to 1.25 inch in outside diameter, and 1/8 inch in thickness. The vacuum pipes contain perforated holes and are wrapped with a geotextile fabric for filtration purposes. The horizontal drainage pipes are typically polyvinyl chloride (PVC) pipes, which are connected to steel vacuum pipes through an elbow connector. Figure 2 shows an array of horizontal drainage pipes connected to vertical vacuum pipes at a project site.

Figure 2 Array of vacuum pipes and horizontal drainage pipes

Step 3: Apply the first cycle of vacuum to reduce water content in the influence zone. In this phase, vacuum-induced dewatering of cohesive soils takes place. Generally, the net effect of this phase of vacuum dewatering is an increase of effective stress up to about 50 to 80 kPa, depending upon the efficiency of vacuum consolidation. It is noted that the highest vacuum pressure that can be exerted on the pore water in the soil is 1 atmosphere pressure (100 kPa). The undrained strength gain of normally consolidated soft clays corresponding to 50- to 80-kPa effective stress increase is roughly 15 to 25 kPa. Therefore, this phase of vacuum dewatering is primarily used for making the site accessible for equipment to carry out the next phase of work (i.e., deep dynamic compaction). The time required for completing this cycle of vacuum consolidation is dictated by the spacing of vertical vacuum pipes and by the horizontal hydraulic conductivity of the soils. In addition, smearing effects (soil disturbances due to installation of vertical vacuum pipes) need to be taken into account. This phase of work is typically completed within seven days before proceeding to the next phase of work.

Step 4: Apply deep dynamic compaction to create a crater and to generate positive pore water pressure. The direct impact from the heavy tamping creates a crater, resulting in displacement of soils and a corresponding reduction in void ratio (direct densification), while producing positive pore pressure in the influence zone. Previous studies indicated that deep dynamic compaction in cohesive soils can cause a rapid increase in both pore water pressure and gas pressure, whether the soil is fully saturated or not, due to the presence of micro air bubbles. The important controlling parameters of dynamic compaction are the weight, dimension, drop height, grid spacing, and number of tamper drops per spots. Decisions regarding these parameters need to be made based on site monitoring results to ensure that the soils underneath the bottom of the crater do not suffer from undrained shear failure or the so-called “rubber soil” phenomenon. A typical dimension for a tamper is about 1 to 1.5 meters in diameter, and the weight can vary from 20 to 70 tons. The tamper drop height varies from 10 meters to approximately 20 meters. A study by Mostafa (2010) provides useful correlations between crater depth, soil properties, influence zones, and tamper energy. The charts presented in Mostafa’s dissertation could be used in the preliminary selection of the controlling parameters. The duration of this phase of work can be accomplished within seven days for a typical 10,000 m2coverage area.

Step 5: Apply the second cycle of vacuum to facilitate a rapid dissipation of pore pressure and to further reduce the water content and void ratio of the soils in the influence zone. The combined efforts of vacuum-generated negative pore water pressure and the deep dynamic compaction generated positive pore water pressure will create a very high pore pressure gradient, which in turn helps to facilitate accelerated dissipation of the pore water pressure, resulting in reduced water content. The duration of this phase is generally seven days or less.

Step 6: Evaluate the soil properties after completing Step 5. In particular, the water content, pore pressures, ground water elevation, ground subsidence, and in-situ test results (such as cone resistance of CPT or N values of STP), need to be determined to assess the results of the first cycle (Steps 4 and 5) of the HVDM process. Evaluation of the outcome of ground improvement at this stage would allow for adjusting the operational parameters (spacing and depth of vacuum pipes, dynamic compaction energy level and grid spacing of tamping points, etc.) in the next cycle of the HVDM process.

Step 7: Repeat Steps 4 through 6 until the performance criteria are satisfied. It should be noted that, in general, two cycles of HVDM process are typically sufficient to achieve the required performance criteria, such as the strength (as determined by CPT or STP) and the post-treatment settlement.

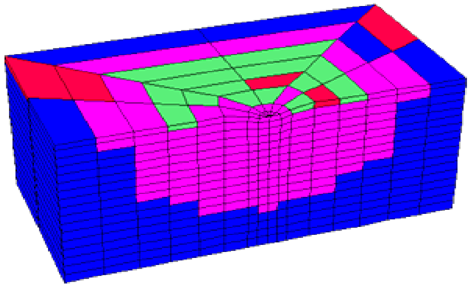
HVDM utilizes the combination of active drainage, consolidation, and densification principles. In order to be successful, this ground improvement method needs to be executed in a controlled manner based on the feedback obtained using on-site monitoring data.

In summary, the aforementioned method is a repeated process that uses a combination of vacuum consolidation/dewatering and dynamic compaction, applying increased tamper impact energy with each successive cycle to achieve the desired density and the required depth of treatment. In addition, on-site monitoring plays a key role to ensure that not only the soil properties before and after ground improvement are monitored but also that the operational parameters (e.g., the spacing of vacuum pipes, the energy of the tamper, and the duration between each stage of vacuum consolidation) are optimized.

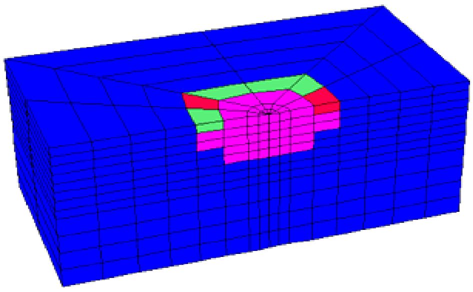
3. TECHNOLOGICAL INNOVATION

The HVDM method embodies at least four technological innovations that are worthy of mention. First, the HVDM method successfully utilizes an intelligent combination of cycles of well-designed vacuum dewatering and dynamic compaction to create not only a very high pore water pressure gradient to expedite pore pressure dissipation, but also to provide active drainage conduits through an innovative, airtight vacuum pipe system. With this generation of high pore pressure gradient and the ability to shorten the pore water drainage path, the HVDM technology essentially extends the applicable range of a vacuum well drainage method into highly impermeable soils with permeability in the order of 1×10^{-7} cm/sec.

The second distinctive innovation of the HVDM method is that it breaks the barrier limiting the use of dynamic compaction in soft, saturated cohesive soils. Dynamic compaction can be applied to advantage in saturated soft clay when used in combination with vacuum well dewatering. The vacuum well dewatering is effective in reducing water content in cohesive soils to the point where the degree of saturation is about 75% to 85%. Therefore, dynamic compaction can be executed in such a way as to avoid the "rubber soil" phenomenon. As shown in Figure 3, a finite element simulation of equivalent static loading on cohesive soil deposits with saturation levels of 100% and 75% indicated that there is a significant difference in the volume of the plastic zone (shown as light shaded area). With the reduction of degree of saturation in the cohesive soils down to 85%, HVDM effectively captures the advantages of dynamic compaction while avoiding its limitations.



(a)



(b)

Figure 3 Finite element simulation results of stress contours under simulated load for (a) S = 100% and (b) S = 75%

The third distinctive feature of the HVDM is the actual densification achieved due to dynamic compaction, which in turn creates a very hard and over-consolidated top layer with thickness in the order of 3 to 4 meters. As illustrated in Figure 4, the presence of this hard, over-consolidated clay layer serves as an effective stress diffuser to spread the surface load with a wider angle of alpha. Therefore, the stresses transmitted to the underlying soil layer are reduced, which would place less stringent requirements on the soil improvement for this underlying soil layer.

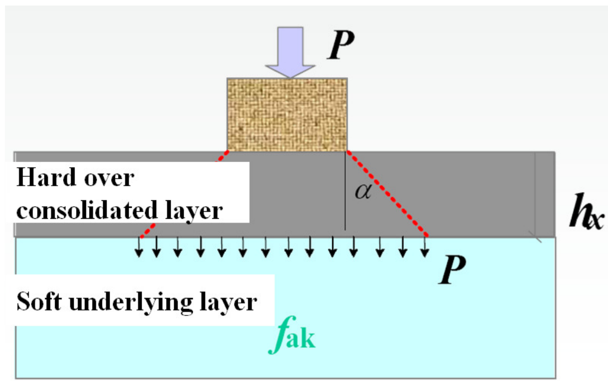


Figure 4 Effective stress distribution due to hard, over consolidated top clay layer

The fourth distinctive feature hinges on the ability to retrieve the vacuum pipes during and after the ground improvement at the site. With the production of hard, over-consolidated clay, which is essentially impervious, in conjunction with the retrieval of vacuum pipes (in contrast to leaving the PVDs in place), the post treatment water drainage path is restricted to the vertical direction and very long horizontal direction (see illustration in Figure 5). As a result, even with additional pore pressure generation due to surface structure loads, the rate of pore pressure dissipation under this restrictive drainage condition would be very slow, thus reducing the rate of post-treatment total and differential settlements.

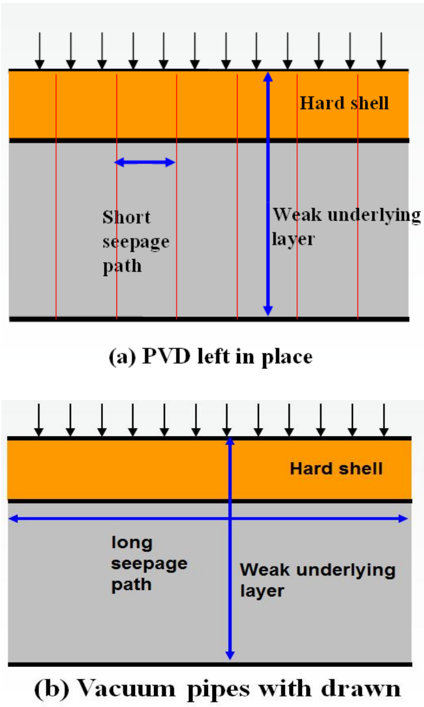


Figure 5 Comparison of drainage path: (a) PVD left in place and (b) vacuum pipes withdrawn

4. ADVANTAGES AND LIMITATIONS

Some of the interesting advantages of using a combination of vacuum consolidation and dynamic compaction include (a) enhancing the vacuum well drainage techniques in fine grained soils with relatively low permeability due to the enhanced pore pressure gradient created by combining the dynamic compaction-induced positive pore pressure and vacuum-induced negative pore pressure, (b) overcoming the common obstacle that dynamic compaction could not be applied to saturated cohesive soils due to the ability to lower the groundwater table and the creation of unsaturated soil zones through vacuum dewatering/consolidation, and (c) expediting pore pressure dissipation due to the creation of a high pore pressure gradient as the result of the combination of negative vacuum pore pressure and positive pore pressure generated by the dynamic compaction. The anticipated results include the following: (a) the creation of a highly over-consolidated clay layer near the ground surface with a thickness in the range of 5 to 8 meters depending upon the deep dynamic compaction efforts and the influence zone (see explanations in the next section), and (b) eliminating the post-treatment horizontal drainage path as a result of withdrawing the vacuum pipes from the ground after completion.

The limitations of the vacuum consolidation/dynamic compaction method are not without limitations. The treatment depth is generally limited to 10 m due to the limit of the influence zone of deep dynamic compaction and the loss of vacuum efficiency when exceeding that depth. In addition, fine grained soils that contain a large portion of organic materials would not be suitable for this method due to the pronounced secondary compression (creep) that could not be treated. The range of permeability of the fine grained soils that can be suitable for this treatment method is limited to a minimum of about 10-6 cm/sec.

5. MECHANISMS OF UNDRAINED STRENGTH GAIN

The mechanisms of the combined vacuum consolidation and dynamic compaction (DC) in improving the soil strength and reducing water content can be illustrated by the *e-log p'* plot in Figure 6, where e is the void ratio and p' is effective mean stress. It is assumed that the fine grained soil is normally consolidated and the state prior to improvement is at Point A. With dynamic compaction, positive excess pore pressure is generated, moving the soil state from Point A to Point B1 (ignoring the apparent reduction of void ratio due to dynamic compaction). Subsequent to dynamic compaction, high vacuum is used to dissipate excess pore pressure rapidly to bring the soil state from Point B1 to Point D1. The reduction of the void ratio is due to dissipation of excess pore pressure during the accelerated consolidation process. With repeated cycles of vacuum and dynamic compaction, both the density and undrained strength of the fine grained soils can be improved. The undrained shear strength of the improved fine grained soils can be estimated by relating the undrained shear strength to the apparent over-consolidation ratio (OCR) as follows.

$$OCR = \frac{p'_{Ei}}{p'_{Ai}} \tag{1}$$

$$\frac{(S_u)_{OCR}}{(S_u)_{NC}} = OCR^{\Lambda} \tag{2}$$

where Λ is an empirical constant.

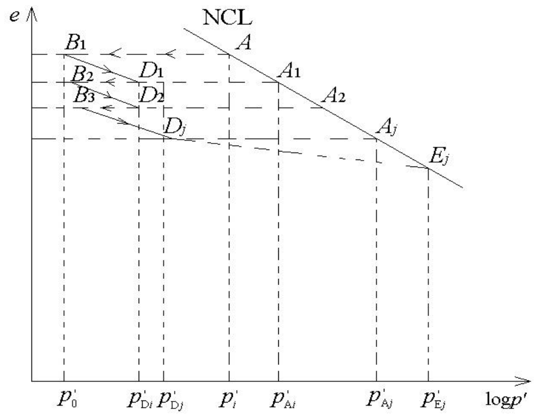


Figure 6 An *e-log p'* plot showing multiple cycles of dynamic impact and vacuum consolidation

6. DISTINGUISHING FEATURES OF HVDM

By using a high vacuum system and adjusting the compaction parameters, the water content in the soil can be reduced. This creative use of a high vacuum system effectively overcomes conventional objections for using dynamic compaction in saturated soft soils. Furthermore, with the sequenced and repeated cycles of vacuum dewatering and deep dynamic compaction, HVDM can successfully treat soils with low permeability within a significantly shortened duration. HVDM produces a hard shell of up to 5 to 8 meters in thickness on the surface of the treated ground, which serves as an excellent load-bearing layer and an impervious seepage barrier. The hardened and impervious shell effectively diffuses the surface loads and impedes drainage of water from soils underneath the hardened surface layer, thus effectively reducing post-treatment consolidation rate (if any) while minimizing post-treatment total and differential settlement.

7. OPTIMIZATION

The success of the a system for using a combination of vacuum consolidation and dynamic compaction for soil improvement depends upon the utilization of field monitoring data collected as part of the QA/QC process. This field monitoring data typically includes measurement of pore water pressure, ground water level, crater depth, ground subsidence, water content, and CPT. This data allows for optimizing the operational parameters, including the heavy tamping energy (by adjusting the mass of tamper, height of drop, spacing and number of drops per spot) and the vacuum consolidation parameters (such as spacing and depth of vacuum pipes).

8. CASE STUDIES

The following two case studies demonstrate the successful application of HVDM in soil improvement projects.

8.1 Case 1

The first case is a land reclamation project at Ningbo Port, China, with an objective to provide a site for coke storage with the intended storage capacity up to 5 million tons per hectare of area. The soil profile at the site consists of a 2-m layer of clay, underlain by a 2.0-m layer of hydraulically filled fly ash, and underlain by a 4.0-m mud clay layer, which is underlain by silty sand. The first phase of treatment area is about 300,000 square meters with the requirement that the improved site provides the bearing pressure up to the range of 30 to 40 kPa. Pictures of site conditions before and after ground treatment using the HVDM ground improvement method are shown in Figure 7. Typical comparisons of CPT cone resistance results before and after ground improvement are shown in Figure 8.

As part of this project, a test program was conducted in an area designated as Site B having four subdivisions labeled as B11, B12, B21, and B22. The vacuum pipes and PVDs arrangements in each zone are summarized in Table 1. The controlling parameters for deep dynamic compaction in each subdivision are presented in Table 2. It should be noted that in all subdivisions, the third stage of dynamic compaction was carried out with 800 to 1000 kN-m of impact energy, with the number of drops equal to two, and with no spacing between the drop spots. This final run of dynamic compaction is commonly referred to as the "final ironing."

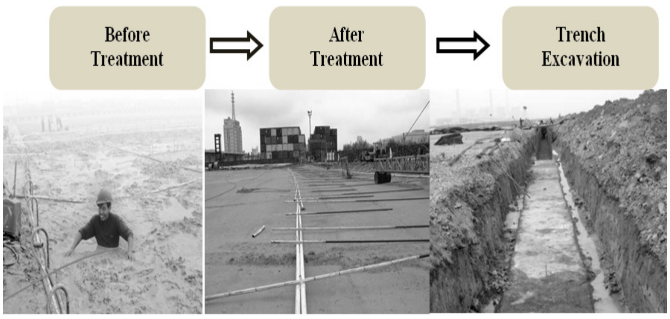


Figure 7 Photos of site conditions at Ningbo Port before and after ground improvement

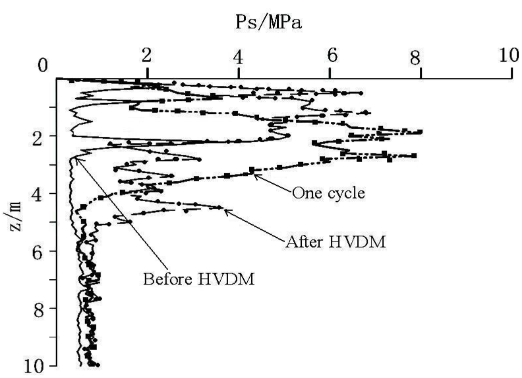


Figure 8 CPT cone resistance readings before and after ground improvement

During the experimental program, several characteristics of the site response were monitored. The surface elevation was measured with a 5-m by 5-m grid. Pore pressure sensors were installed at depths of 3.5 m and 6 m. A groundwater observation well at 4 m below the surface was installed and was monitored twice per day. The water content in the soil was measured before and after each stage of dynamic compaction. Monitoring of vacuum pressure was also performed at the test site. As part of evaluation of the soil properties, static cone penetration tests were conducted.

8.1.1 Analysis of Monitoring Results at the Test Site

Surface settlement

The settlement for Subdivisions B11 and B12 at the end of each of the first, second, and third cycles of dynamic compaction was 42 cm, 17 cm, and 6 cm, respectively. For Subdivisions B13 and B14, the settlement at the end of each of the first, second, and third cycles was 35.3 cm, 29.5 cm, and 8.9 cm, respectively. In all cases, the first stage of dynamic compaction contributed the most settlement toward the accumulated total surface settlement.

Pore pressure monitoring results

Representative pore pressure response is shown in Figure 9. From this figure, it can be seen that the pore pressure dissipation rate is very high when vacuum is applied. With the pore pressure gradient generated by vacuum and dynamic compaction, dissipation of pore pressure occurred very rapidly. For the first and second DC/vacuum run, 90% of the generated pore pressure had been dissipated within 7 to 8 days. Both vacuum pipes and PVDs were used for drainage.

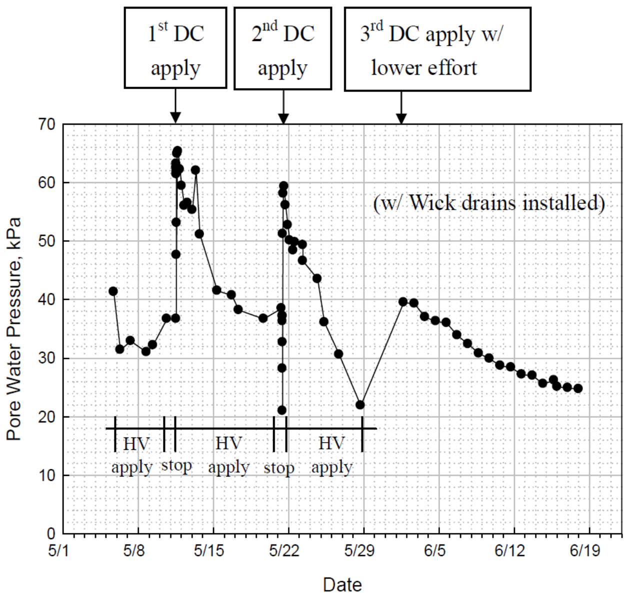


Figure 9 Pore pressure response with vacuum pipes and PVDs

Water content

In the hydraulically filled fly ash layer, the water content was reduced from 54.7% to 39.9%, with an average of 15% reduction. In the clay mud layer, the water content was lowered from 53% to 36%, with an average of 17% reduction. In the silty sand layer, the water content did not change. Therefore, it was concluded that the method can reduce water content down to 5 to 6 meters from the ground surface. The method would not affect the water content in the soil layer that is 10 m or deeper from the ground surface.

Evaluation of Improvement Results

Subdivisions B11 and B13 were subjected to higher impact energy and a lower number of impacts, while Subdivision B13 had PVDs installed. Subdivisions B12 and B14 were subjected to lower impact energy but a larger number of drops. Subdivision B14 had PVDs installed. The average improvement of the entire Site B is as follows. In Layer 2, cone resistance was increased from 0.74 MPa to 2.51 MPa, with an improvement ratio of 3.37. In Layer 3, the cone resistance was increased from 0.21 MPa to 0.35 MPa, with an improvement ratio of 1.66. In comparing Subsections B13 to B11 or B14 to B12 (i.e., zones with PVDs and zones without PVDs), it can be seen that the improved cone resistance can be on average 10% to 20% higher in zones with PVDs than in zones without PVDs.

8.2 Case 2

The second case is an on-going HVDM project in Indonesia that is located in southern Sumatra. The geomorphic features of the site are mainly low hills and plains. A portion of the site will be leveled by cutting and the remaining area will be backfilled with the cut soil material. Based on the soil investigation report, the shallow subsoil can be described as heavy silty clay, wet to saturated, ranging from soft plastic to hard plastic. Particle gradation analysis indicates about 8% to26% clay particles, 6% to 38% sand particles, and 37% to 69% silt particles. Photos of heavy silty clay collected as core samples are provided in Figure 10. The cone penetration test results are summarized in Table 3.

Table 1 Arrangement of vacuum pipes and PVDs

Area	PVD (8 m long) Spacing	Vacuum Pipes (6 m deep) Spacing	Vacuum Pipes (3 m deep) Spacing
B11	1.1 m × 1.1 m	3.5 m × 5.0 m	3.5 m × 2.25 m
B12	1.1 m × 1.1 m	3.5 m × 5.5 m	3.5 m × 2.75 m
B13	1.1 m × 1.1 m	3.5 m × 5.0 m	3.5 m × 2.25 m
B14	1.1 m × 1.1 m	3.5 m × 5.5 m	3.5 m × 2.75 m

Table 2 Controlling parameters for dynamic compaction (DC)

Area	Spacing	1st Stage of DC		2nd Stage of DC	
		Energy per drop (kN·m)	Number of Drops	Energy per drop (kN·m)	Number of Drops
B11	4.0 m × 4.0 m	800	3	1200	2
B12	4.0 m × 4.0 m	1200	2	1600	2
B13	4.0 m × 4.0 m	800	3	1200	2
B14	4.0 m × 4.0 m	1200	2	1600	2

Table 3 Cone penetration test results for heavy silty clay

Soil stratum	Item	Cone tip resistance (MPa)	Side friction (kPa)	Frictional ratio (%)
1-8 Heavy silty clay (hard plastic)	Number for statistics	120	105	118
	Maximum	20.00	300.0	6.3
	Minimum	1.20	50.0	0.3
	Average	11.25	139.6	1.9
	Standard deviation	10.49	129.8	1.7
	Coefficient of variation	0.431	0.420	0.793

Figure 10 Heavy silty clay cores

Due to the fact that a portion of the cutting and backfilling work was conducted during the raining season, and the water content of backfilled silty clay site was significantly high, as shown in Figure 11. Subsequently, HVDM was used to improve the uncompacted silty clay soils. Photos of the construction site during HVDM soil improvement work are shown in Figure 12. Although the project is still on-going, some of the preliminary performance test results from the plate load tests indicated that the modulus of elasticity is in the range of 27 Mpa, while the settlement at 4 mm corresponding to the applied pressure of 1.5 Kg per centimeter square. Also, the dynamic cone penetration test results indicated that the blow count is 6 to 7 down to depth of 4 m, then increasing to a blow count of 12 around 4 meter and 6 meter depth. The in-situ silty clay soil was successfully improved to meet the performance criteria.

9. NUMERICAL SIMULATION OF HVD

Numerical simulation of soil response to dynamic compaction in combination with vacuum consolidation in saturated and partially saturated soils should be done very carefully. In this work, free fall kinematic energy of a tamper transferred to soil surface through Mohr–Coloumb interface was modeled. In addition, soil properties, such as strength and stiffness, were updated during impact in every five time steps. For modeling large deformation beneath tamper, mesh coordinates were updated in every five time steps.

10. SOIL CONSTITUTIVE LAW

In this study, a modified Cam–Clay model is used for modeling soils. The modified Cam–Clay model is an incremental hardening/softening elasto-plastic model. Its features include a particular form of nonlinear elasticity and a hardening/softening behavior governed by volumetric plastic strain. The failure envelopes are self-similar in shape and correspond to ellipsoids of rotation about the mean stress axis in the principal stress space. The shear flow rule is associated flow rule. In the Cam–clay model, it is assumed that any change in mean pressure is accompanied by an elastic change in volume according to Eq. (3); therefore, the tangent bulk modulus of the Cam-Clay material calculated from Eq. (4):

$$\Delta p = \frac{vp}{\kappa}\Delta\varepsilon_p^e \tag{3}$$

$$\mathrm{K} = \frac{vp}{\kappa} \tag{4}$$

where p is the mean effective pressure, v is the specific volume, κ is the swelling line slope, and K is the bulk modulus.

10.1 Yield Function

The yield function corresponding to a particular value p_c of the consolidation pressure has the form:

$$f(q,p) = q^2 + M^2 p(p - p_c) \tag{5}$$

where M is a material constant, the slope of the critical state line; p_c is the consolidation pressure; q is the deviatoric stress; and p is the mean effective pressure.

10.2 Hardening/Softening rule

The size of the yield curve is dependent on the value of the consolidation pressure, p_c. Eq. (6) shows cap hardening rule for Modified Cam-Clay.

$$\frac{dp_c}{de^p} = \frac{p_c}{\lambda - \kappa} * (\nu_\lambda - \lambda * \ln(\frac{p_c}{p_{ref}})) \tag{6}$$

It has 4 parameters λ, κ, υ_λ and P_{ref}.

10.3 Cam–Clay Baseline Parameters

The Baseline Cam–Clay parameters used in this research are summarized in Table 4 and they are representative of a normally consolidated clay with undrained shear strength of 7 kPa at a depth of 2 m.

Figure 11 Uncompacted heavy silty clay

Figure 12 Site construction photo

Table 4 Modified Cam–Clay parameters

M	λ	κ	Reference Pressure (kPa)	Reference Specific Volume (υ_λ)	Density, ρ (Kg/m^3)	K_0
1.0	0.1	0.04	1	2.5	1500	0.6

11. DYNAMIC COMPACTION SIMULATION CONSIDERATIONS

For modeling dynamic compaction, a 3-D model is employed to assess the response of soil mass during impact. Note that the mesh used consists of polyhedral elements. Mesh size or element size should be carefully selected for the problem of wave propagation in a semi-infinite half space. On one hand, the very small element size can cause numerical instability; on the other hand, large elements cannot allow the shorter waves (which are associated with highest frequencies) to travel. The time increment is selected so that the primary waves, which are the fastest, can be recorded on two consecutive nodes along the travel distance. In this research using Flac3D computer program, the dynamic time step should be less than the critical dynamic time step calculated by Eq. (7), and the mesh size should be less than Eq. (8).

$$\Delta t_{crit} = \min\left\{\frac{V}{C_p A^f_{\max}}\right\} \tag{7}$$

$$\Delta l \leq \frac{\lambda}{10} \tag{8}$$

where C_p, V, and $A^f_{\max}$ are the p-wave speed, the tetrahedral subzone volume, and the maximum face area associated with the tetrahedral subzone, respectively. The variable λ is the wavelength associated with the highest frequency component that contains appreciable energy.

The dynamic time step of 1×10^{-6} sec for the rigid body impact modeling has been used by others. Since this value is less than the critical dynamic time step of *Flac3D*, it is selected as a dynamic time step through the present study. The mesh size is chosen as 0.35 m which is smaller than the critical mesh size. The top view of 3-D finite difference mesh including dimensions and boundary conditions and its elevation view are presented in Figures 13 and 14, respectively. DC impacts could be modeled by applying a force-time history or modeling the real impact, by considering the tamper as a shell structural element and defining shear and normal parameters of interface for modeling impact between the tamper and the soil surface by initializing free fall velocity to tamper grid points (nodes) as the initial condition of simulation.

In this research, the Mohr–Coulomb interface was employed for modeling real impacts. The interface parameters are summarized in Table 5.

A good rule-of-thumb (provided by *Flac3D*documentation) is that k_n and k_s be set to ten times the equivalent stiffness of the stiffest neighboring zone. The apparent stiffness (expressed in stress-per-distance units) of a zone in the normal direction is:

$$\max\left\{\frac{(K+\frac{4}{3}G)}{\Delta z_{\min}}\right\} = \max\left[\frac{(5(Mpa)+\frac{4}{3}*2(Mpa))}{0.25m}\right] \tag{9}$$
$$= 5*10^7 \frac{N}{m^3}$$

where, K & G are the bulk and shear moduli, respectively; and $\Delta z_{\min}$ is the smallest width of an adjoining zone in the normal direction. In this research, k_n taken as 8×10^8 N/m^3 and k_s=2×10^8 N/m^3.

12. SIMULATION RESULTS OF HVDM

Figure 13 shows a simulated pattern of HVDM for illustration purpose. It is assumed to be conducted in square pattern for both vacuum pipes and dynamic compaction drop points. Since simulating the effect of all dynamic compaction drops on one vacuum pipe is impossible and unnecessary; thus, we consider that each vacuum pipe is affected only by the four impact points around it.

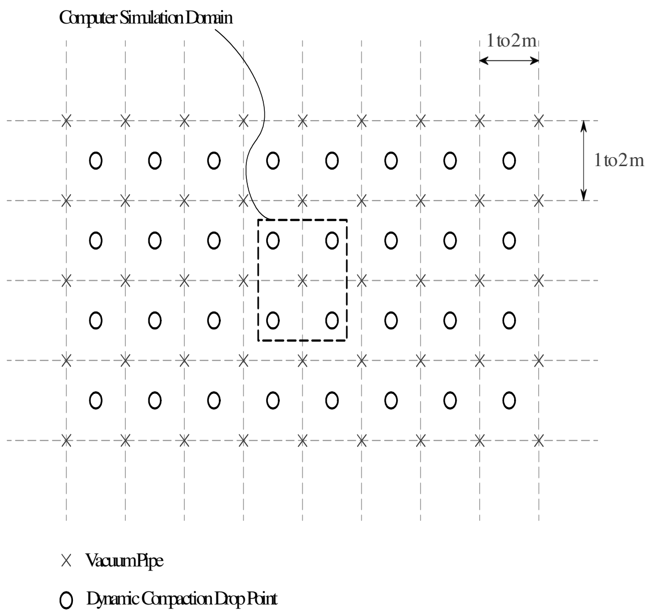


Figure 13 General pattern of HVDM

Figure 14 shows that dynamic compaction was performed in four drop points successively. Dynamic compaction treatment in this simulation consists of three impacts per location. The impact grid points were spaced at 3.0 m in a square pattern. The tamper radius was 1.0 m and its weight was 8 tons. The drop height was selected as 10 m. The hydraulic conductivity value of 10^{-9} m/sec was selected for this simulation. The coefficient of volume compressibility, m_v, was estimated as 0.6 (MPa^{-1}) according to available literature data for normally consolidated soft clay for a representative element at depth of 3 m, corresponding to a mean effective stress of 50 kPa. According to Eq. (10), c_v, k_h and m_v are correlated with each other; therefore, c_h is equal to 5.25 m^2/year.

$$c_h = \frac{\kappa_h}{\gamma_w * m_p} \tag{10}$$

where c_h is the horizontal coefficient of consolidation; k_h is the horizontal hydraulic conductivity; γ_w is the water density; and m_v is the coefficient of volume compressibility.

Based on numerical simulation results, variations of excess pore water pressure through a vertical section from Drop Point 1 to Drop Point 3 are shown in Figure 15, due to repeated impacts. It can be seen from these results that excess pore water pressure was generated with accumulation of impacts at one drop point and then with impacts at the neighboring drop points.

Table 5 Mohr–Coulomb interface parameters for modeling real impact

Shear coupling cohesion (kPa)	Shear coupling friction angle (deg.)	Shear coupling stiffness (N/m^3)	Normal coupling cohesion (kPa)	Normal coupling friction angle (deg.)	Normal coupling stiffness (N/m^3)
7	10	2×10^8	7	10	8×10^8

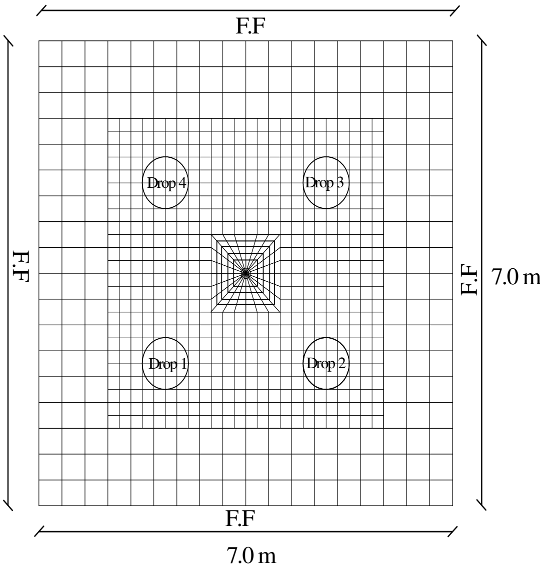


Figure 14 Top view of generated mesh

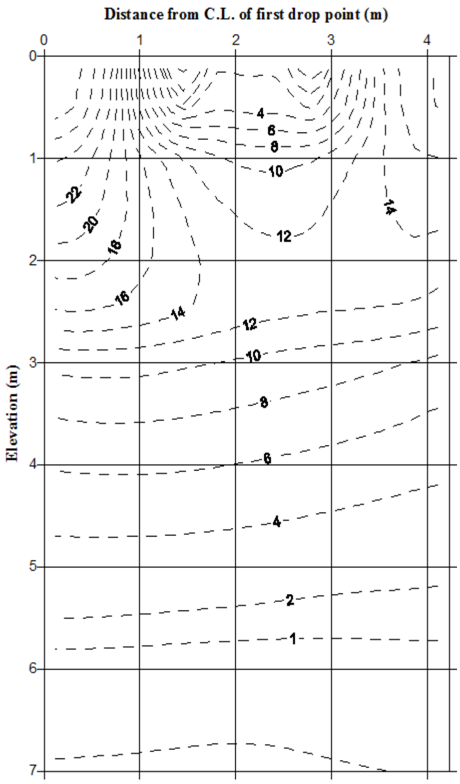


Figure 15 Contour of generated EPWP after finishing all drops around vacuum pipe

Figure 16 and Figure 17 illustrate contours of the generated EPWP inside soil mass for the first and the third tamper drops. As it can be seen, with increasing drop number, the influence zone of EPWP was extended laterally and vertically. It should be mentioned that initial degree of saturation of the soil was selected as 85% in the numerical simulation. Figure 18 shows the vertical stress contours. Figure 19 and Figure 20 show the vertical displacement and horizontal displacement contours, respectively.

Through the above illustration, numerical simulation results support some of the mentioned HVDM mechanisms.

Figure 20 shows the contour of vertical displacement. From this figure, it can be seen that close to the surface and around the edges of the tamper, upheaving occurred. The maximum amount of upheaving is 0.04 m.

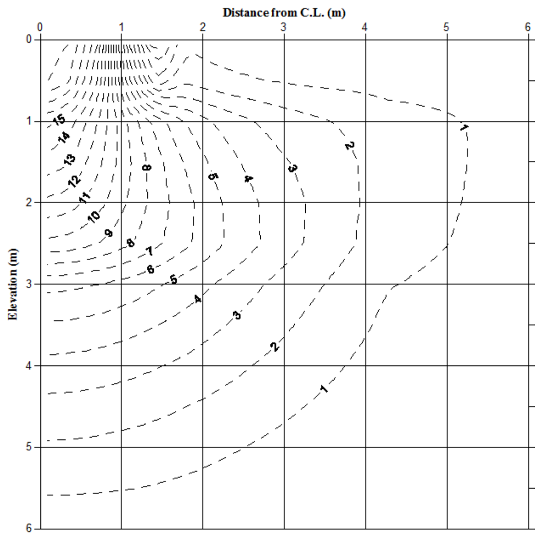


Figure 16 Contours of generated EPWP (kPa) for Drop 1, with an initial degree of saturation of 85%

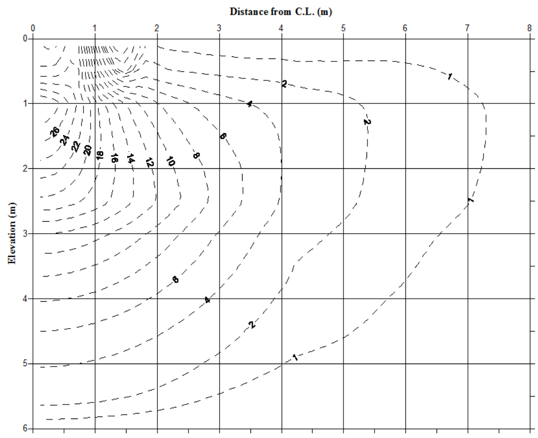


Figure 17 Contours of generated EPWP (kPa) for Drop 3 with an initial degree of saturation of 85%

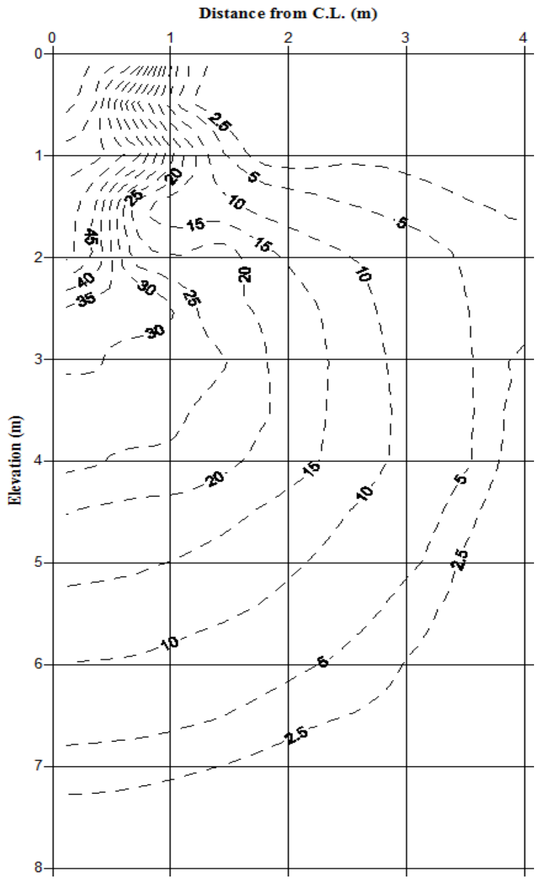


Figure 18 Contours of total vertical stress

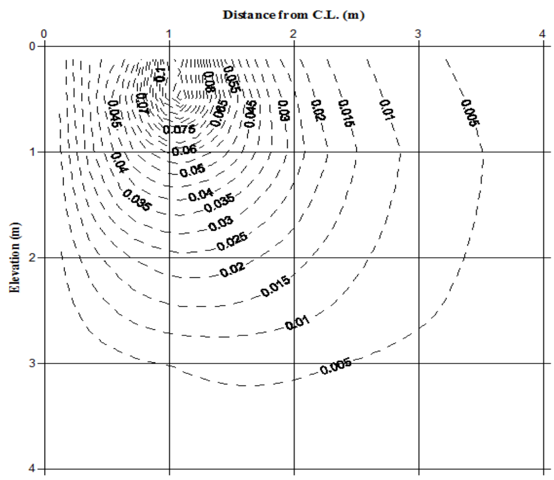


Figure 19 Contours of horizontal displacement

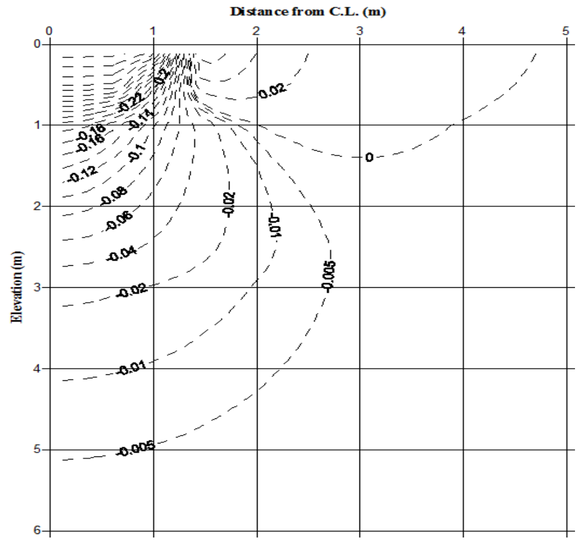


Figure 20 Contours of vertical displacement

13. SUMMARY AND CONCLUSIONS

In this paper, recent advances in ground improvement techniques for fine grained soils involving the combined use of vacuum consolidation/dewatering and deep dynamic compaction were described. Specifically, the working principles of this method and its advantages and limitations were discussed. A case study presented the monitoring and evaluation results of a pilot testing program at the Ningbo Port Project in China. The site monitoring data confirmed the working principles of the described ground improvement method. The water content was effectively reduced and the undrained shear strength indicated by cone resistance was increased as a result of the application of this ground improvement method. A second case study presented an on-going project in Indonesia, where HVDM was used to improve silty clay soils. While this project is not yet completed, preliminary results indicate that the clay soil was improved sufficiently to meet performance criteria. Three-dimensional numerical simulations of HVDM using FLAC3D program indicated the positive pore pressure generation due to dynamic compaction in nearly saturated fine-grained soils. Specific conclusions are as follows.

1) The mechanisms of HVDM technology for ground improvement are as follows: Dynamic compaction in nearly saturated contractive soils can generate positive pore water pressure, which when coupled with vacuum induced negative pore pressure, can expedite drainage of water from the soil mass, thus rapidly increasing density and undrained shear strength of the treated soil.
2) Field monitoring data and FLAC3D numerical simulation results, presented in this paper, provided supporting evidences for the aforementioned mechanisms of the HVDM soft soil improvement technology.

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