DESIGN AND ANALYSIS OF “HIGH VACUUM DENSIFICATION METHOD” FOR SATURATED AND PARTIALLY SATURATED SOFT SOIL IMPROVEMENT

A Dissertation
Presented to
The Graduate Faculty of The University of Akron

In Partial Fulfillment
of the Requirement for the Degree
Doctor of Philosophy

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May, 2014
DESIGN AND ANALYSIS OF “HIGH VACUUM DENSIFICATION METHOD” FOR SATURATED AND PARTIALLY SATURATED SOFT SOIL IMPROVEMENT

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Dissertation

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ABSTRACT

One of the main challenges that geotechnical engineers and contractors encounter is the In-situ improvement of soft cohesive soils. The importance of doing this improvement fast and economically clarifies in countries with large population and, consequently, large request of infrastructures, such as China, India, and other developing countries in Asia. The traditional techniques to improve soft cohesive soils are: (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 (g) deep dynamic compaction. Although all of the above-mentioned techniques are available in field of soil improvement, but PVDs in combination with fill surcharge preloading method appears to be the most common used technique. Recently, the vacuum consolidation with surcharge preloading looks interesting for expediting consolidation and reducing consolidation time. In large projects like land reclamation of the dredged materials, port facility constructions, airport runways, soft soil improvement area could be considerably large, resulting in the limitation on the availability of surcharge material for fill preloading. Hence, development of more economic ways to treat soft cohesive soils in large projects, where preloading fill could not be economically found, is of interest.

In the past, the application of vacuum either alone or in combination with static surcharge loading has been utilized to facilitate consolidation in saturated cohesive soils.
Though the dynamic compaction technique was introduced by Menard (1975) as a feasible technique for use in saturated cohesive soils, due to possible rubber soil phenomenon it has not been widely used to improve soft cohesive soils. Fast development of infrastructures in China leads to an innovative soft cohesive soil treatment technique in 2000, rapidly applied in advancing countries in Asia. The core of the above-mentioned treatment method has been termed as “High Vacuum Densification Method” (HVDM). This technique combines two well-known soil improvement methods consisting of vacuum consolidation and deep dynamic compaction and makes an intelligent yet efficient soft soil treatment method suitable for large areas within a relatively short time period.

The work presented throughout this research is concerned with developing a numerical modeling of HVDM in saturated/partially saturated cohesive soil using FLAC 3D with Cap-Yield and Modified Cam-Clay constitutive models with coupling fluid and mechanical together in order to achieve a better understanding of the behavior of cohesive soils under dynamic compaction and vacuum consolidation. An extensive FD parametric study conducted to investigate the influence of soil properties, operational parameters of DC, and negative vacuum pressure amplitude on the soil responses like: generation of EPWP, change of mean effective stress and surface settlement. Finally a step by step design methodology for HVDM in saturated and partially saturated cohesive soils will be presented.
ACKNOWLEDGEMENTS

I am heartily thankful to my supervisor, Prof. Robert Liang, whose encouragement, guidance and support from the initial to the final level enabled me to develop an understanding of the subject. I gratefully offer my regards and blessings to all of those who supported me in any respect during the completion of the project. Last but not the least; I would like to thank my family: my parents, for giving birth to me at the first place and supporting me spiritually throughout my life and my sisters and brother.
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NOTATIONS

W  Tamper Weight
H  Drop height
D  Tamper radius
A  Tamper Area
ν  Poisson’s ratio
e  Void ratio
k  Permeability
g  Element Size
λ  Wave length
f  Frequency
κ  Logarithmic elastic bulk modulus
λ  Logarithmic plastic bulk modulus
a₀  Initial yield surface size
M  Slope of critical state line
DC  Dynamic compaction
FD  Finite difference
EPWP  Excess pore water pressure
CL  Center line
CHAPTER I

INTRODUCTION

1.1 Overview

One of the main challenges of geotechnical engineers is ground improvement of soft cohesive soils. In emerging populated countries like China and India, infrastructure construction is growing up; therefore the need for developing fast and affordable soft soil improvement methods is necessary. The most common techniques of soft soil improvement comprise the application of the following methods: (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. Liang R. (2012) through the various above mentioned techniques, the method of using vertical drains in combination with fill surcharge preloading seems to be the most applicable method throughout the world. Recently in some projects for reducing consolidation time, vacuum surcharge applied to the soil mass in addition to fill surcharge. In large projects like reclamation of the dredged materials, petro-chemical plants, etc. the treated area is very large and affording fill surcharge is not economical. Therefore, a new technique developed for treating soft cohesive soils in large areas where using fill surcharge is expensive.

A new soft cohesive soil treatment technique was developed in China in 2000, the core of this cohesive soil treatment method was termed as “High Vacuum Densification
Method (HVDM)**, and this fast soft soil treatment technique combines two well-known soil treatments methods, vacuum consolidation and dynamic compaction. so that soils at the project site can be improved through the effects of lowered water content and increased density. As a result, soil strength and stiffness are improved. Furthermore, total and differential settlements after HVDM treatment are minimized.

1.2 History of HVDM

The development of HVDM returned to early 2000, when the inventor, Mr. Shi-Long Xu of Shanghai Geoharbour Group, conducted some experiments for assessing concept of high vacuum densification method and using this technique in future projects. After successful applications in China, HVDM was applied in other countries in Asia, like Vietnam, Malaysia, and Indonesia. Currently, HVDM has become most common method used for land reclamation in China, with more than 10 million meter square of land treated in the last 10 years.

1.3 Working Principles of HVDM

HVDM is a fast soil treatment technique combining drainage, consolidation, and densification principles. HVDM typically performed in a controlled manner based on feedback of on-site monitoring data for both QA/QC purpose. Figure 1.1 shows schematic drawing of HVDM. This method consists of the following steps:
Step 1: performing soil investigation at the field and determining soil profile at the site. Some of relevant soil parameters such as soil particle size distribution, Atterberg Limits, water content, hydraulic conductivity, compressibility, and coefficient of consolidation should be estimated and determined, in addition some in-situ tests like CPT or STP should be conducted for establishing baseline properties before starting HVDM in the field.

Step 2: installing vertical perforated vacuum pipes and horizontal drainage pipes. Vacuum pipes typically have an outside diameter of 1 to 1.25 inch, and 1/8 inch in thickness. The horizontal drainage pipes are typically PVCs, which are connected to steel vacuum pipes through an elbow connector. Figure 1.2 shows an array of horizontal drainage pipes connected to vertical vacuum pipes at a project site.
Step 3: Apply first cycle of vacuum for reducing water content and initial degree of saturation, in addition increasing load bearing capacity of soil, therefore soil can tolerate weight of labors and equipment’s. In this phase dewatering occurred through induced vacuum and mean effective stress increase up to about 50-80 kPa. It should be mentioned that the upper limit of vacuum pressure which can be exerted to the soil mass is 100 kPa. Independent of stress path and history of loading, undrained shear strength, $Su$ is related to mean effective stress and OCR, over-consolidation ratio, and with a rough estimation, the undrained strength gain of normally consolidated soft clays corresponding to 50 to 80 kPa effective stress increase is approximately 15 to 25 kPa. The vacuum consolidation time for this cycle is a function of vertical drain spacing and horizontal soil permeability. This phase is fulfilled usually within one week before proceeding to the next step.

Step 4: Applying dynamic compaction for increasing mean effective stress inside soil mass and generation of excess pore water pressure. Designing parameters of DC are tamper weight and diameter, drop height, spacing between drop points, and number of
tamper drops per spots which should be considered very carefully. In this step design as you go method, instead of predictive method can be applied for correcting primarily DC design according to site monitoring results to ensure that the soils underneath the bottom of the crater do not suffer undrained shear failure or the so called “rubber soil” phenomenon. The duration of this phase is usually within 1 week for an area of approximately 10,000 square meters.

Step 5: Apply the second cycle of vacuum for dissipating generated non-uniform EPWP during DC efforts, and reducing degree of saturation and void ratio of soil mass. The combined efforts of vacuum generated negative pore water pressure and the deep dynamic compaction generated positive pore water pressure create very high pore pressure gradient, which in turn help facilitate accelerated dissipation of pore water pressure, resulting in reduced water content. The duration of this phase is generally one week or less.

Step 6: Assessing soil responses, such as the water content, pore pressures, ground water elevation, and ground subsidence after finishing step 5, in addition some in-situ tests such as CPT or STP should be conducted to evaluate results of first cycle (Steps 4 and 5) of treatment. Assessing the outcome of ground improvement at this step would allow for adjusting the operation parameters (spacing and depth of vacuum pipes, dynamic compaction energy level and grid spacing of tamping points, etc.) in the next cycle of HVDM process.

Step 7: Repeating Steps 4 through 6 for satisfying the performance criteria. It should be mentioned that in general two cycles of HVDM process are generally sufficient
to achieve the required performance criteria, such as strength as determined by CPT or STP and the post-treatment settlement.

1.4 Distinguishing Features of HVDM

Through the use of high vacuum system and adjustment of compaction parameters, the water content in the soil can be reduced. This creative use of high vacuum effectively overcomes the conventional reluctance in 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 meter 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) with the beneficial results of minimized post-treatment total and differential settlement.

1.5 Advantages and Limitations of HVDM

Technical breakthrough of HVDM includes: (a) extending the vacuum well drainage to fairly impermeable cohesive soils, (b) overcoming the common notion that dynamic compaction could not be applied to saturated cohesive soils, and (c) expediting pore pressure dissipation due to creation of high pore pressure gradient. The results of HVDM include the following particular end products: (a) creation of a highly over-consolidated clay layer on the upper portion of the ground with thickness in the range of
5 to 8 meters depending upon the deep dynamic compaction efforts and the influence zone, (b) eliminating the post-treatment drainage path due to withdrawal of vacuum pipes from the ground after completion and creation of a fairly impervious soil layer on the ground surface, which is contrast to the conventional PVDs that would have to be left in the ground. The limitation of HVDM include that the treatment depth cannot exceed 10 meter due to the limit of influence zone of deep dynamic compaction and loss of efficiency for vacuum dewatering exceeding that depth. In addition, cohesive soils contain large portion of organic materials may not be suitable for HVDM. The range of cohesive soils for HVDM is fine grained soils with hydraulic conductivity not less than $5 \times 10^{-7}$ cm/sec.

1.6 QA/QC Process

The success of HVDM depends upon intelligent utilization of field monitoring of relevant information to allow for optimization of HVDM operation parameters, including heavy tamping energy (mass of tamper, height of drop, spacing and number of drops per spot) and vacuum consolidation parameters, such as vacuum pipes spacing and depth, among others. Field monitoring typically includes measurement of pore water pressure, ground water level, crater depth, ground subsidence, water content, and CPT (or SPT).

1.7 Summary and Conclusions

In this research, recent advances in soft clay improvement techniques using principles of vacuum dewatering and deep dynamic compaction were described and analyzed with Flac3D software. Specifically, a series of invention that is commonly referred to as “High Vacuum Densification Method (HVDM)” was described in this research. The technology of the HVDM soft cohesive soil improvement method has been
advanced in China in recent years, with its rapid expansion into the neighboring countries to provide economical soft soil improvement method with savings in both construction time and construction cost. The advantages of the HVDM method, compared to the conventional prefabricated vertical drains (PVD) and surcharge preloading techniques are as follows: (a) the vertical vacuum pipes are re-usable and can be easily connected to the horizontal PVC pipes and the vacuum pumps to form a closed vacuum system for dewatering, (b) there is no need to bring in surcharge load as the dynamic compaction provides the means of generating positive pore water pressure and mechanical densification of the soft clays, and (c) the combined negative pore pressure from vacuum and positive pore pressure from dynamic compaction facilitate expeditious dissipation of pore pressure and resulting fast consolidation of the clay soil deposit, (d) there is no need for a special construction requirement to carry out the work in the field, which is contrast to the necessity of a special equipment to install PVDs, and (e) compared to PVD, vacuum pipes function as a better conduit for vacuum and dewatering. Finally, the many successful applications have provided evidences of the simplicity and effectiveness of the HVDM methods in improving the soft cohesive soil sites. The method can also be considered as a green technology, as it involves no use of chemical additives.

1.8 Problem statement for High Vacuum Densification Method

As you know Dynamic Compaction works best on deposits where the degree of saturation is low, deposits for which dynamic compaction is not appropriate include saturated clayey soil (either natural or fill). In saturated deposits, improvement cannot occur unless the water can be expelled from the voids. In clayey soils where the permeability is low, the excess pore water pressure generated during dynamic compaction
require a lengthy period of time to dissipate, for solving these problems in saturated clay deposits, a new methodology developed by Chinese researchers, this method called HVDM, High Vacuum Densification Method, that is using vacuum preloading and dynamic compaction in combination for saturated cohesive deposits, in cohesive soils permeability of the soil mass is very low for rendering dynamic compaction practical for these deposits, degree of saturation and drainage length should be decreased, because of that, in HVDM system, PVD, wick, installed in soil mass for transferring negative vacuum pressure and reducing drainage length as well, in HVDM, at first step, 1 cycle of high negative vacuum pressure, say -70 kPa, applied to soil mass for decreasing degree of saturation and after that 1 cycle of dynamic compaction, these combination iterated successively till reaching to target results. For designing HVDM following parameters should be considered:

- Profile of degree of saturation after each cycle of vacuum pre-loading
- Vertical and lateral spreading of excess pore water pressure generated after dynamic compaction
- Change of soil properties like, permeability because of compaction
- Optimum time of consolidation, time needed for dissipation of EPWP for each cycle of vacuum pre-loading

1.9 Objectives

The objectives of this research are as follows:

- Develop a numerical modeling of HVDM in saturated or partially saturated cohesive soil using FLAC 3D with Cap-Yield constitutive model with coupling fluid and
mechanical together in order to achieve a better understanding of the behavior of cohesive soils under dynamic compaction and vacuum consolidation.

- Develop a step by step design methodology for HVDM in saturated and partially saturated cohesive soils
- Validate the developed FD modeling techniques using existing field and/or lab test data
- Conducting extensive FD parametric study to investigate the influence of soil properties and operational parameters of DC on the outcome of D.C. process and pwp response for vacuum consolidation phase.
- Assessing group effect of vacuum pipes, for vacuum pre-loading, how water table or degree of saturation profile change, when we have a group of vacuum pipes.
- Performing parametric study on negative vacuum pressure with considering 4 different vacuum pressure: -40kPa, -50kPa, -60kPa and -75kPa
- Correlating permeability to soil properties, the coefficient of permeability, $k_w$ is a function of void ratio, $e$, degree of saturation, $S_r$, and water content, in drainage phase change of permeability should be considered.

1.10 Motivation

Here I summarize motivations and missing of my research area

- Lack of numerical simulation of dynamic compaction on saturated and partially saturated cohesive soil
- Lack of well accepted design methods for D.C. in cohesive soil
- Lack of parametric study on D.C. parameters in saturated and partially saturated cohesive soils like: EPWP, Influence zone, Grid Space, Optimum energy, Crater Depth
• There isn’t any numerical simulation that consider change of water bulk modulus with Sr and PWP during compaction and in most hydro-mechanical coupled researches water bulk modulus considered as a maximum pure water bulk modulus

• The coefficient of permeability, is a function of void ratio, e, degree of saturation, Sr, and water content, in drainage phase change of permeability should be considered but in most simulations permeability considered constant

• accessing vacuum consolidation for non-uniform generated PWP is a new work

• Current Design models for DC are simplified and cannot consider elastic and plastic modulus change during compaction

• Most numerical models are not hydro-mechanical coupled therefore they couldn’t model pore water pressure generation

• Most of design methodology are for granular soils like sand, not for saturated or partially saturated cohesive soils
CHAPTER II

DYNAMIC COMPACTION & VACUUM CONSOLIDATION LITERATURE REVIEW

2.1 Deep dynamic compaction

Deep dynamic compaction (DC) is repeated application of dropping heavy weight, tamper, on the ground surface for densifying soil deposits. Although this technique origins from Romans era, but Menard (1975) explained its fundamentals as a technique for ground densification in the late 1960s; in the literature this technique also known as heavy tamping, dynamic consolidation, deep DC too, Green R. (2001). The mass of the tamper typically ranges from 5.4 to 27.2 tons, and drop height ranges from 12.2 to 30.5\( m \), Green R. (2001). DC usually performed in phases. The first phase (high energy phase) uses heavy masses dropped from higher elevations repeatedly with greater distance between drop points. Initial high energy phase is usually followed by a low energy phase, called "ironing", the goal of this phase is densifying the surficial layers in the upper 1.5\( m \). ASCE (1997) Figure 2.1 shows this soil improvement method schematically.
Figure 2.1 Schimatic picture of deep dynamic compaction. (Adapted from Green 2001).

2.2 Technique Suitability

Range of soil particle-size distribution suitable for dynamic compaction shown in Figure 1-2, in this figure, deposits categorized to 3 categories: Zone 1, Zone 2, Zone 3, which soils that their sieve analysis curve lie at Zone 1 being the most suitable for DC and soils with particle-size distribution at zone 3 are the last suitable for DC.

With increasing DC application, the soil types treated by this technique have also become more various. Initially, the most suitable soil types for dynamic compaction treatment were granular deposits with low degree of saturation and high permeability, like ballast fills or natural sandy gravelly soils only. But because of affordable nature of
this method, various types of deposits, with low conductivity and high initial degree of saturation, like, silty sand, soft clay and collapsible soils have been treated by this method. However, this technique works best on material where the initial degree of saturation is low and permeability of the soil mass is high.

Figure 2.2 Grouping of soils for dynamic compaction. Zone 1 soils are most suitable for deep dynamic compaction. (Adapted from Lukas 1986).
2.3 Fundamentals of Dynamic Compaction

Menard and Broise (1975) explained 4 steps taking place during dynamic compaction soil improvement:

1- Compressibility of saturated deposits due to the presence of micro-bubbles;
2- The gradual transition to liquefaction under repeated impacts;
3- Dissipation of excess pore water pressure due to permeability after soil fissuring;
4- Thixotropic recovery.

2.3.1 Compressibility

Fine saturated soils, like clay, classified as incompressible when abrupt shock, like dynamic impact in dynamic compaction, applied to them, their low conductivity don’t allow rapid dissipation of excess pore water pressure, This drainage of pore water pressure considered to be a necessity for allowing settlement of soil due to Terzaghi consolidation theory, Menard (1975). But early observations showed that in saturated fine soils, dynamic compaction created an immediate considerable crater at soil surface, but traditional theories considered fine saturated soil incompressible, future works showed that natural water inside soil deposits contained, micro-bubbles and entrapped air pockets in the voids between soil particles, the air-content change between 1% and 4% Menard (1975).

2.3.2 Liquefaction

When energy transferred from tamper to soil under the form of cycling loading in dynamic compaction, dissolved air inside water will be compressed gradually, and water bulk modulus will increase too, therefore gradually soil behave as an incompressible
material, and liquefaction of soil will happen at this step. Menard (1975) called the energy level required to reach this stage as a saturation energy. Menard and Broise (1975) developed a special apparatus, called dynamic oedometer for measuring amount of this energy. Fig. 2-3 illustrates this instrument. It should be mentioned that when amount of accumulated dissipated energy reached to this saturated energy, more application of energy will be damped and excess pore water pressure will not increase more.

Figure 2.3 Menard Dynamic Oedometer Apparatus

2.3.3 Permeability

During performing dynamic consolidation projects, it is observed that, excess pore water pressure drained very quickly which couldn’t be explained by permeability measured before impact, Menard (1975). During tamper impact, generated high pore pressure create large hydraulic gradient, and this gradient is sufficient for tearing soil tissue, and creating fissures, this fissures propagate through stress concentration in newly created cracks, and increase soil mass permeability after impact.
2.3.4 Thixotropic recovery

during dynamic impact soil structure disturbed and shear strength of soil decrease, but with passing time, generated excess pore water pressure due to dynamic compaction will dissipate and soil tissue will setup, therefore soil shear strength and stiffness will increase too, this phenomenon called as Thixotropy or aging. Different stages of dynamic compaction process shown in Figure 2.4, after several drops of tamper on soil surface liquefaction will be induced, moreover after dissipation of excess pore water pressure soil strength will increase.

Figure 2.4 Stage of soil conditions as a result of successive passes of deep dynamic compaction. (Adapted from Menard and Broise 1975).
2.4 Practices of Dynamic Compaction

Dynamic compaction usually performed in square grid pattern, and typically 5 to 15 blows dropped at every drop point, grid space usually lie between 5 to 10m, moreover depth of improvement usually selected as thickness of weak layer that should be treated Green (2001).

Because of excessive settlement of soil surface and deep crater depth, or need for time for dissipation of generated excess pore water pressure, number of successive drops should be limited, and drops will be conducted in several passes. A pass defined as specified sequential number of blows for every drop point. After performing each pass created crater will be filled with granular soil, like sand and gravel, before starting next pass.

Dynamic compaction sometimes performed in phases, in first phase high energy tamping employed for densifying deeper layers, for avoiding creation of hard crust during first phase distance between drop points should be selected far enough; because of disturbance of surficial soil during initial phase, high energy phase followed by second phase that called ironing phase, the goal of this phase is compacting upper (5ft) layer. Figure 2.5 shows effects of different phases on soil strength, as it can be seen, surficial layers lose their strength during high energy phase, but after second phase, upper layer strength will be improved. In addition, this figure shows depth of maximum improvement too, and it is defined as the depth at which maximum improvement of soil properties achieved.
Figure 2.5 The influence of the high energy and ironing phases of dynamic compaction and the effect of thixotropy. Adapted from Green (2001).
2.5 Vertical and Lateral Improvement, zone of influence

The effective depth of improvement (DI) considered as the maximum depth at which significant change in soil mechanical properties, like Dr, volumetric strain, $S_u$ and etc., will take place, a lot of works done previously for correlating the depth of improvement (DI) to operational parameters of tamping, like W, tamper weight, H, drop height, and etc. The most practical equation for calculating DI, is Eq. (2-1)

$$DI = n. \sqrt{W.H}$$

(2-1)

Where:

DI = Maximum depth of improvement (m).

n= Empirical constant

W= Mass of tamper (tones: 1tonne=1Mg).

H = Drop Height (m).

In this equation, the term $W.H$ is a measure for gravitational potential energy of the tamper at its free fall height. Figure 2.6 shows different values of $n$ with combination of actual field data. Smith and De Quelerij (1989) presented a one dimensional soil model to compute the extent of the plastic zone (influence depth) based on impact velocity, tamper mass, contact area, and elastic soil properties as

$$DI' = \frac{m}{\rho_{max} A} \left[-1 + \left(\frac{\sigma_e + aV^2}{\sigma_e}\right)^{0.5}\right]$$

(2-2)

Where:

DI’: net influence depth (DI less the crater depth)

$\rho_{max}$: maximum possible soil density

m: falling mass

A: contact area
σe: vertical stress at elasticity limit

V₀: Impact velocity

α: density ratio

Luongo (1992), based on 30 set of field data, suggested Eq. (2-3) for calculating depth of improvement

\[ DI = k_1 + k_2(W \cdot H) \]  

(2-3)

Where \( k_1 \) and \( k_2 \) are regression parameters. Lucas (1986) explained parameter’s that affect DC design. These parameters divided into two parts:

a) site related parameters:
   - Soil deposit type
   - Existence of hard layer above or beneath the improved soil
   - Existence of softer layer beneath the improved soil
   - Initial degree of saturation of soil

b) equipment related parameters:
   - Tamper parameters like: tamper weight, radius and drop height.

All of the above mentioned factors should be considered for estimating DI and grid spacing between drop points in initial design of DC.
Figure 2.6 Relation between depth of influence and tamper momentum W.H. Adapted from Green (2001).

The value of n is a function of Lukas (1995):

- How much is efficient, tamper dropping by crane
- How much is WH, energy that applied by weight
- Soil type being compacted.
- Presence of energy damping layers, like soft soil deposits
- If a hard layer existed above or below the soil layer being compacted, it will affect value of n.

- Dynamic stress applied to the soil surface during impact.

Generally, n will increase as the soil permeability increase. Table 2.1 shows different values of n for various soil deposits.

Table 2.1 Suggested value of n from different scholars. Adapted from (Green 2001).

<table>
<thead>
<tr>
<th>Reference</th>
<th>$n$ - values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Menard and Broise (1975)</td>
<td>1.0</td>
</tr>
<tr>
<td>Lenards et al. (1980)</td>
<td>0.5</td>
</tr>
<tr>
<td>Ramaswamy et al. (1981)</td>
<td>0.6*</td>
</tr>
<tr>
<td>Bhandari (1981)</td>
<td>0.51</td>
</tr>
<tr>
<td>Charles et al. (1981)</td>
<td>0.35*</td>
</tr>
<tr>
<td>Santoyo and Fuentes (1982)</td>
<td>0.37</td>
</tr>
<tr>
<td>Bjolgerud and Haug (1983)</td>
<td>1.0 (rockfill)</td>
</tr>
<tr>
<td>Smolteczyk (1983)</td>
<td>0.5 (soils with unstable structure)</td>
</tr>
<tr>
<td></td>
<td>0.67 (silts and sands)</td>
</tr>
<tr>
<td></td>
<td>1.0 (purely frictional soils)</td>
</tr>
<tr>
<td>Lukas (1984)</td>
<td>0.65 – 0.8</td>
</tr>
<tr>
<td>Mayne (1984)</td>
<td>0.3 – 0.8</td>
</tr>
<tr>
<td>Gambin (1984)</td>
<td>0.5 – 1.0</td>
</tr>
<tr>
<td>Qian (1985)</td>
<td>0.55 (loess)</td>
</tr>
<tr>
<td></td>
<td>0.65 (fine sand)</td>
</tr>
<tr>
<td></td>
<td>0.66 (soft clay)</td>
</tr>
<tr>
<td>Van Impe (1989)</td>
<td>0.5 (clayey sand)</td>
</tr>
<tr>
<td></td>
<td>0.65 (silty sand)</td>
</tr>
</tbody>
</table>

*Values computed by Moreno et al. (1983) from data in listed reference.
Poran (1992) investigated vertical and lateral extent of DC, in this work it is assumed that plastic zone has a semi spheroid shape, and its dimensions correlated to accumulated input energy, the research performed on dry sands by a small scale specimen. Equations (2-4) and (2-5) show the dimensions of the plastic zone as a function of the normalized energy input.

\[
\frac{b}{D} = j + k \log \left( \frac{NWH}{Ab} \right) \tag{2-4}
\]

\[
\frac{a}{D} = l + m \log \left( \frac{NWH}{Ab} \right) \tag{2-5}
\]

Where:

- a: spheroid base radius
- b: spheroid height
- D: diameter of falling tamper
- N: number of drops
- W: falling weight
- H: falling height
- A: tamper area
- J, k, l, m: regression constants

Chow (1994) investigated the lateral extent of DC operation by conducting cone penetration tests. For correlating sand friction angle to CPT test results, Meyerhof’s equations were used. The change of friction angle due to dynamic compaction process explained by equation (2-6), he concluded that at distance three times the tamper diameter the influence of the DC isn’t considerable. He also concluded that when dynamic compaction performed on spaced grid pattern the middle of grid sides are points of weakness.
\[
\frac{\Delta \phi}{\Delta \phi_b} = 0.642 - 1.180 \log\left(\frac{X}{D}\right)
\]  
(2-6)

Where:

\(\Delta \phi\): change of friction angle at distance X from drop point C.L.

\(\Delta \phi_b\): change of friction angle at the C.L. of tamper due to DC

D: tamper diameter

X: distance from C.L. of tamper

Lukas (1995) explained that the depth of maximum improvement usually lie between 0.33DI to 0.5DI, where DI is depth of influence, Green R. A. (2001) mentioned that this depth coincide with depth of maximum lateral displacement, he also concluded that this coincidence is because of large deviatoric shear strain that happen at this depth, therefore lateral deflection could be a good measure for checking quality of densification.
Oshima and Takada (1997) considered relative density, $D_r$, as a measure for assessing dynamic compaction influence zone; they supposed that densified area has a bulb shape with depth ($Z$) and radius ($R$); note that $Z_{10}$ and $R_{10}$ means depth and radius of area with 10% increase in relative density, respectively. They also concluded that depth ($Z$) and radius ($R$) of the compacted area have linear relations with logarithm of total momentum of impacts during DC.
2.6 Crater depth

During tamper impact on top of soil surface soil beneath tamper will settle, and a crater will be created, usually around the edges of crater, soil will heave, crater depth regarded as a measure for degree of improvement. Typical values of crater depth lie between 0.5m to 1.5m. Pan J.L. (2002) explained that deeper crater depth means that energy transferred to soil mass over longer impact dynamic time, therefore the peak particle acceleration of outgoing waves diminish quicker, and the depth of influence is reduced too. Mayne (1984) performed a study, to illustrate that with increasing impact energy crater depth will increase too. Solocombe (1993) applied value of cater depth to the depth of influence as a measure for soil suitability for dynamic compaction, Table 2.2 shows crater depth as a percentage of depth of influence.

Table 2.2 Crater depth vs. depth of influence for different soil types (Solocombe 1993)

<table>
<thead>
<tr>
<th>Soil Category</th>
<th>Soil Type</th>
<th>Crater Depth (as a percent of DI)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone 1</td>
<td>Natural Sand</td>
<td>3-10</td>
</tr>
<tr>
<td></td>
<td>Granular Fill</td>
<td>5-15</td>
</tr>
<tr>
<td>Zone 2</td>
<td>Semi pervious soils</td>
<td>N/A</td>
</tr>
<tr>
<td>Zone 3</td>
<td>Clay fills</td>
<td>3-5</td>
</tr>
<tr>
<td></td>
<td>Natural Clay</td>
<td>1-3</td>
</tr>
<tr>
<td>Other</td>
<td>Refuse and Peat</td>
<td>7-20</td>
</tr>
</tbody>
</table>

Rollins et al (1994) assessed effect of number of drops on increasing crater depth, they concluded that with increasing number of drops crater depth will not increase considerably and after 6-8 drops, crater depth will not progress, Oshima (1994) analytically assessed effect of falling weight impact energy on crater depth, he concluded that crater depth is a function of falling weight momentum. He conducted some centrifugal model test for dynamic compaction for multiple drops, The model soil sample was a loose Toyora sand with a relative density of 50% and water content of 4%,.
acceleration of centrifuge is 100g and tamping applied inside centrifuge box, one side of the centrifuge box is made of glass therefore created crater depth measured through this glass side, they concluded that crater depth is a function of square root of drop numbers, Moreover Oshima (1994) did some field measurement for assessing ram deceleration during impact, A tamper with mass of 25 tons and base area of 4m² was dropped from heights 1, 2, 5 and 10 meters, the tamper acceleration always exhibits a half sinusoidal shape. The contact time between the tamper and soil last for 0.050 sec and is not sensitive to tamping energy level, as shown in Figure 2.8

![Figure 2.8 Deceleration history of tamper for different drop height (Oshima A. 1994)](image)

2.7 Energy and Propagated Waves

Green R. A. (2001) used term of cumulative potential energy per unit area of field for assessing density of applied energy, if this term divided by thickness of improved layer it can be converted to saturation energy (J/m³) for estimating needed saturation energy for the project. Eq. (2-7) explains this relation.
\[ AE = \frac{N \cdot W \cdot H \cdot P \cdot g}{A_{cp}} \]  

(2-7)

Where:

\( AE \) = Applied energy (kJ/m\(^2\)).

\( W \) = Mass of tamper (tones: 1tonne=1Mg).

\( H \) = Drop height (m).

\( P \) = Number of passes.

\( N \) = Number of drops per pass.

\( g \) = acceleration due to gravity (9.81m/sec\(^2\)).

\( A_{cp} \) = Tributary area per compaction (m\(^2\)).

Lukas (1995) prepared Table 2.3 for estimating required energy per unit volume for compacting various soil deposits.
Table 2.3 Guideline for selecting required energy per unit volume of various soil types. Lukas (1995)

<table>
<thead>
<tr>
<th>Type of Deposit</th>
<th>Unit Applied Energy (kJ/m$^3$)</th>
<th>Percent Standard Proctor Energy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pervious coarse-grained soil (Zone 1)</td>
<td>200 - 250</td>
<td>33 - 41</td>
</tr>
<tr>
<td>Semi-pervious fine-grained soils (Zone 2) and Clay fills above the water table (Zone 3)</td>
<td>250 - 350</td>
<td>41 - 60</td>
</tr>
<tr>
<td>Landfills</td>
<td>600 - 1100</td>
<td>100 - 180</td>
</tr>
</tbody>
</table>

Note: Standard Proctor energy equals 600 kJ/m$^3$

In the practical projects Usually Eq.2-1 used for estimating DI, depth of influence, DI usually lie in the range of 5 to 8m in typical projects, for reaching to higher values of DI in special cases, heavier tamper and more powerful crane is needed. Figure 2.9 shows relationship between tamper weight and drop height for several different projects.
Figure 2.9 variations of tamper mass and drop height gathered from different projects. Green (2001).
Table 2.4 that presented by Lukas (1986) is a practical guideline for selecting crane size and cable size, for different ranges of tamper weights.

Table 2.4 Guideline for selecting crane size and cable size. Lukas (1986).

<table>
<thead>
<tr>
<th>Tamper Weight</th>
<th>Crawler Crane Size</th>
<th>Cable Size (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.5 to 7.5Mg</td>
<td>40 to 50 tons</td>
<td>19 to 22</td>
</tr>
<tr>
<td>7.5 to 13Mg</td>
<td>50 to 100 tons</td>
<td>22 to 25</td>
</tr>
<tr>
<td>13.5 to 16.5Mg</td>
<td>100 to 125 tons</td>
<td>25 to 29</td>
</tr>
<tr>
<td>16.5 to 23Mg</td>
<td>150 to 175 tons</td>
<td>32 to 38</td>
</tr>
</tbody>
</table>

Green R. A. (2001) explain that for an elastic medium, during dynamic compaction, impact energy carried by 3 types of waves, P-, S-, and Rayleigh waves, the portion of energy transferred by these waves shown in Figure 2.10 for sinusoidal vertical loading, the percentage of energy transferred by every type of waves is a function of dimensionless frequency parameter $a_0$:

$$a_0 = \frac{\omega r_0}{v_s}$$  \hspace{1cm} (2-8)

Where:

- $a_0$ = Dimensionless frequency.
- $\omega$ = Frequency of applied loading (rad/sec).
- $r_0$ = Equivalent radius of tamper and radius of proctor mass.
- $v_s$ = shear wave velocity of the soil.
2.8 Wave Transmission limitations during dynamic compaction

Whenever a tamper impact soil surface, outgoing waves carried out of site, higher impact momentum causes higher vibrations, therefore if project located in a residential area, neighbor structures maybe affected by dynamic impact waves, In the codes and dynamic compaction guidelines, particle velocity used as a parameter for limiting ground vibrations, to avoid damage to neighbor structures, The U. S. Bureau of Mines presented Figure 2.11 based on several dynamic compaction projects, frequency of outgoing waves during dynamic compaction lie in the range of 6 to 10 Hz. FHWA (1999), in the Figure 2.11 it can be seen that particle velocities should be limited to 13 and 19 mm/sec for older and newer buildings, preventing cracks in the walls, project measurements show that considerable damages doesn’t take place until particle velocities become more than 50 mm/sec.
In a dynamic compaction project, particle velocity can be measured with seismograph, and the value should be less than thresholds suggested in Figure 2.11 on the soil surface neighbor to the building.

In advance of performing dynamic compaction project, the particle velocities should be estimated, to check that particle velocity limitations will be exceeded or not? Fig. 2.12, FHWA (1999) developed based on many projects measurements for estimating soil particle velocity in various soil deposits, the horizontal axis is the scaled energy factor, that is a measure of attenuated impact energy reach from impact drop point to the point of concern, and the vertical axis is soil particle velocity.
If dynamic compaction performed close to neighbor buildings, site should be isolated from existing facilities, and outgoing waves should be damped, one of the best methods for diminishing ground vibration is excavating a trench with depth of nearly 3.0m as a barrier between the drop point and neighbor structures, excavated trench shouldn’t interfere with neighbor facilities foundations, most suitable trench for damping of outgoing waves is an open trench, as there isn’t any medium for wave transmission, but if there are some concerns about falling of trench wall or lateral deflection of neighbor structures, trench should be filled by materials with high capability of damping, like loosely placed sand or gravel.

Figure 2.12 Soil particle velocity vs. scaled energy factor for various soil types Adapted from FHWA (1999)
2.9 DC parameters important in design methodology

Finding the soil layers of site by soil investigation and distinguishing the degree of suitability of DC for field deposit as the first issue are the most important parameters in the success of DC. A design methodology would be good if the following DC parameters were selected, tamper weight, and tamper height, the number of passes, and the number of phases. Also, the parameters including soil parameters and the target depth of improvement should be determined for designing DC. Soil parameters consist of conductivity of soil, the initial degree of saturation, drainage length, and soil layers like hard or weak layers above or beneath depth of improvement should be investigated. The best results in DC are achieved on ground deposits with the low degree of saturation, and the high conductivity of soil. The most suitable soils for DC are permeable granular soils. Pervious granular deposits are consisting of sands and gravels, as well as fill deposits including granular mine spoil deposits, building rubble, industrial waste fills. When the situation of deposits is above the water table, densification happens as immediate as the compaction of soil particles into a denser state. When this situation is below the water table, inasmuch as the permeability of the soils is usually high the generated EPWP due to tamper impact dissipated immediately, resulting the immediate improvement. When soil layers are saturated, improvement cannot be reached unless the water can be drained out from the voids. For the clayey soils with low permeability, a long period of time is required for the dissipation of the EPWP generated during DC. The gap between granular permeable soils and saturated soft cohesive deposits is known as the third category of soils named semi pervious soils, like Silts and sandy silts. DC can also be applied for the third category of soils, but since their permeability is not high enough for dissipation of
generated EPWP DC should be conducted in several phases. This process of dissipation can take times in order of days to weeks. To overcome this problem the path of drainage is shortened using wick drains. In some other projects, wick vacuum preloading is also used to decrease the time of consolidation, named as High Vacuum Densification Method (HVDM).

2.10 Calculation of induced Stresses during Impact using numerical methods

There are analytical studies in the literature about the examination of the dynamic stresses induced during DC. One of these works is the study of Scott and Pierce (1975) via the stiffness and damping properties of penetration.

2.10.1 Scott and Pierce approach

Scott (1975) studied the dynamic properties of the soil media like acoustic wave impedance and other effective factors on the initial stress level and thus the deceleration of the tamper. They concluded that how the body of impacted zone area reacts to the movement of tamper at the surface will control the subsequent behaviors including impact duration, the decay of surface dynamic stress, and the crater depth generated by tamper. In addition they explained that dry and well compacted soil would behave in elastic manner, while very loose dry soil would react as an energy absorbing damper. The approximation of the behavior of saturated soils is more difficult in comparison with the other previous soil types since high transient EPWP can destroy the rigidity of the soil matrix and permit a possibility of an impulsive plastic flow and displacement. Via the consideration of simple models for idealized soil it would be possible to illustrate the nature of these various constraints mechanisms as:

- Ideal elastic unsaturated soil
• Elasto-plastic unsaturated soil
• Saturated soil

2.10.2 M. Gunaratne method for calculating dynamic stresses

Gunaratne M., et al (1996) performed some experimental tests for assessing dynamic stress during dynamic compaction. They concluded that vertical and horizontal stress contours in soil mass are the same as for an elastic medium. Figure 2.13 shows that there is a good agreement between Gunaratne M. results and other researchers that used elastic theory. Therefore, if \( \Delta \sigma_0 \) is dynamic stress beneath tamper with radius \( r_0 \), then maximum vertical dynamic stress at any depth \( z \), below of tamper CL can be calculated by elastic theory as:

\[
\Delta \sigma_v = \Delta \sigma_0 \left[ 1 - \frac{1}{1 + \left( \frac{r_0}{z} \right)^2} \right] \tag{2-9}
\]
Figure 2.13 Distribution of vertical dynamic stress below tamper CL compared with other researchers results for elastic theory. Gunaratne M. et al.

2.11 Mechanism of Energy Dissipation in Soils

Due to the wave propagation inside soil mass apportion of energy will be damped, which results in diminish of wave amplitude. The frictional sliding at grain-to-grain contact surface is the dominant mechanism of energy dissipation within the cohesionless soils, (Whitman and Dorby (1993). If the soil would also be saturated, the viscous drag of the pore fluid movement relative to the soil skeleton will lead to the energy dissipation. The other mechanisms like particle breakage don’t have considerable contribution for most soils.
2.11.1 Dissipated Energy

The increment of dissipated energy per unit volume of material, $dW$ can be described as below:

$$dW = \sigma_{ij} d\varepsilon_{ij} \tag{2-10}$$

Where $\sigma_{ij}$ and $d\varepsilon_{ij}$ are the stress and incremental strain tensors, respectively. The expansion of equation (2-10) using Voigt notation leads to

$$dW = \sigma_x d\varepsilon_x + \sigma_y d\varepsilon_y + \sigma_z d\varepsilon_z + \tau_{xy} d\gamma_{xy} + \tau_{xz} d\gamma_{xz} + \tau_{yz} d\gamma_{yz} \tag{2-11}$$

Via the integration of equation (2-11) over an arbitrary stress path, the cumulative energy dissipation per unit volume of material, $\Delta W$ is obtained via the following relation

$$\Delta W = \int dW \tag{2-12}$$

2.11.2 Energy-Based EPWP generation models

There are many studies in the literature which relate excess pore water pressure to the accumulated energy. For example, energy dissipation was related to both of the densification of dry samples and the generation of excess pore pressure in saturated samples by governing differential equations. Nemat-Nasser and Shokooh,  (1979). The theoretical study of Nemat-Nasser and Shokooh was complemented by numerous empirical relations. For instance, as one of these numerical relations the work of Mostaghel and Habibaghi (1978, 1979) named as MH model are presented herein. In MH model

$$r_u = \frac{1}{e_0} \frac{\Delta W}{\sigma_v'} \tag{2-13}$$

In which $\Delta W$, $e_0$, and $\sigma_v'$ are energy dissipation per unit volume of material, the initial void ratio of the soil, and the initial vertical effective stress, respectively.
Throughout this study the accumulated energy dissipation at the stage when the liquefaction of the soil occurs is used to find the saturation energy.

2.12 Induced Excess Pore Water Pressure during Impact

In many dynamic compaction projects water table is close to the soil surface, therefore excess pore water pressure will be generated during impact, but in the literature there aren’t enough research that address generation and dissipation of pore water pressure.

Gunaratne M. et al (1996) assessed generation of excess pore water pressure, due to dynamic impacts on impervious fine soils. They conducted some experimental tests and concluded that dynamic stress distribution with depth is similar to static loading on an elastic medium, in addition similar to Terzaghi theory, they explained that, drainage of generated excess pore water pressure after impact can be vertical, close to the impact region, they applied a modified 1-D Terzaghi theory for predicting excess pore water pressure, equation 2-14 shows weak form of governing differential equation for predicting EPWP.

\[
\frac{\partial u}{\partial t} \left(1 + \frac{e_{CW}}{a_v}\right) = \frac{\partial \sigma}{\partial t}
\]  

(2-14)

Above equation can be simplified in the form of equation 2-15

\[
\frac{\partial u}{\partial t} = B \frac{\partial \sigma}{\partial t}
\]  

(2-15)

Where, B is the Skempton’s pore pressure parameter for isotropic stress increments and explained as below:

\[
B = \frac{1}{(1+\frac{e_{CW}}{a_v})}
\]  

(2-16)
One of the most important conditions regarding suitability of a soil deposit to be treated by D.C. is drainage of excess pore water pressure when water table is close to soil surface. This is due to the fact when there is water inside soil voids, high percentage of dynamic energy converted to excess pore water pressure during impact, and progressive drops will increase pore water pressure until condition that soil become liquefied, Therefore prediction of generated pore water pressure for impermeable or semi pervious deposits is very important.

In FLAC3D’s code, at any node if degree of saturation is less than one pore pressure at that node will be set to zero, therefore for considering the effect of dissolved and entrapped air, water bulk modulus can be reduced while keeping the degree of saturation constant at 1. In addition the initial degree of saturation may be initialized by the user; however it is also updated during FLAC3D’s calculation cycle. It should be mentioned that in FLAC3D, saturation is not an independent variable; it cannot be fixed at any node. In the literature there are some equations for correlating water bulk modulus to initial air bubble percentage and pore water pressure, one of them will be used in present research, therefore water bulk modulus will be updated during running, and generally EPWP is a function of: $EPWP=f(z,r_0,K_f,\Delta\sigma_0,\text{cam-clay parameters})$

Where:

$z$: Depth

$r_0$: Tamper radius

$K_f$: Water bulk modulus

$\Delta\sigma_0$: Dynamic stress
2.13 Change of EPWP with time

Changes of EPWP are illustrated in Figure 2.14. When dynamic wave reach to a point inside soil mass, EPWP will increase abruptly to a high value, due to the passing high impact energy wave. After 0.15 to 0.35 seconds when the velocity of the falling tamper becomes zero, EPWP go down rapidly and reach to a constant value which called residual EPWP. For the time after impact up to the tamper stop, the soil almost experiences undrained conditions. During this very short time, EPWP generation does not depend on the soil conductivity. After this time, consolidation phase begins and the developed EPWP diminish gradually with a permeability-dependent rate and soil will be consolidated. As it is shown in Figure 2.14, EPWP grows suddenly up to a high level and, then, falls abruptly down to zero or negative values.

Figure 2.14 Excess Pore Water Pressure variation with time
2.14 Compressibility of water

Menard and Broise (1975) considered Compressibility of saturated soil due to the presence of micro-bubbles as one of the fundamentals of DC, natural water inside soil deposits contains entrapped air pockets in the voids between soil particles, and these micro-bubbles increase compressibility of air-water mixtures, and therefore decrease generation of excess pore water pressure during impact considerably. For example, Chaney (1978) mentions that the water bulk modulus can be decreased by an order of magnitude for an air-water mixture at 99% saturation in compacted sand. Hence, it is essential to take into account the compressibility of air-water mixture instead of using the compressibility of pure water. Yoshimi et al. (1989) performed some element tests, cyclic torsional shear tests on hollow cylindrical specimens, to study the effect of initial degree of saturation on the underained cyclic shear strength (liquefaction resistance) of air-pluviated Toyoura sand. The tests conducted by Yoshimi et al. (1989) are in good agreement with the theoretical relationship proposed by Lade and Hernandez (1977)

\[
B = \frac{1}{1 + nK_s \left[ \frac{S_r}{K_w} + \frac{1-S_r}{u_a} \right]} \quad (2-17)
\]

where \( n \), \( K_s \), \( K_w \) and \( u_a \) are porosity, bulk modulus of soil skeleton, bulk modulus of pure water (2.23×106 kPa), and absolute pressure in pore fluid, respectively. In this research, Eq. (18) is used for finding equivalent bulk modulus of water, \( K_w' \)

\[
\frac{1}{K_w'} = \frac{S_r}{K_w} + \frac{1-S_r}{u_a} \quad (2-18)
\]

How Eq.(18) is obtained from Eq.(17) is explained in the following. For cohesive fine grained soils, since permeability is low and impact time is small too, we can suppose that analysis is undrained during impact, and water content, \( \omega \) is constant for every element,
during undrained phase. So, from Eq. (2-19) it is found that \((S_r, e)\) is constant during impact. Regarding the initial soil conditions, \(S_r, e,\) and pore water pressure are known. Consequently, we can calculate \(S_r\), degree of saturation in each step from Eq. (2-19), and \(K'_w\) will be recalculated for every step from Eq. (2-18)

\[
S_r e = \omega G_s. \tag{2-19}
\]

Note that, \(e\) and \(G_s\) in Eq. (2-19) are void ratio and specific gravity, respectively. The curve in Figure 2.15 shows that test results, conducted by Yoshiaki Yoshimi and Tanaka, follow the theoretical relationship proposed by Lade and Hernandez (1979), Eq. (2-17)

![Figure 2.15 Relation between degree of saturation and B-value](image)

Figure 2.15 Relation bet. Degree of saturation and B- Value

To simulate unsaturated condition in Flac3D, it is supposed that we have a saturated soil, \(S_r = 100\%\), with an equivalent bulk modulus, reduced bulk modulus, that has the same B-Value of an unsaturated soil. In unsaturated soil, we have three phases, trapped air, pure water, and soil skeleton. Lade and Hernandez (1977) proposed the above-mentioned theoretical equation to calculate B-value,
\[ B = \frac{1}{1 + nK_s \left( \frac{S_r - (1 - S_r)}{u_a} \right)} \]  

(2-17)

where, \( n, K_s, K_w, \) and \( u_a \) are porosity, bulk modulus of soil skeleton, bulk modulus of pure water \((2.23 \times 10^6 \text{ kPa})\), and absolute pressure in pore fluid, relative pressure +1e5Pa, respectively. For saturated sample by considering \( S_r = 1.0 \) and water bulk modulus equal to reduced one, \( K_w' \), Eq. (2-17) is converted to Eq. (2-20) for unsaturated sample.

\[ B_1 = \frac{1}{1 + nK_s \left( \frac{S_r K_w}{K_w'} + \frac{(1 - S_r)}{u_a} \right)} = \frac{1}{1 + nK_s \left( \frac{S_r K_w}{K_w'} \right)} \]  

(2-20)

\( B_2 \) is calculated from Eq. (2-17) as below:

\[ B_2 = \frac{1}{1 + nK_s \left( \frac{S_r}{K_w} + \frac{(1 - S_r)}{u_a} \right)} \]  

(2-21)

\[ B_1 = B_2 \Rightarrow \frac{1}{K_w'} = \frac{S_r}{K_w} + \frac{1 - S_r}{u_a} \]  

(2-22)

Eq. (2-22) is employed to find the equivalent bulk modulus of water, \( K_w' \) at the end of each step.

2.15 Physical model tests

Wetzel and Vey (1970) performed a series of model tests for measuring stress and strain inside soil mass, during impact of tamper on soil surface, for this purpose they used Ottawa sand and concluded that distribution of vertical stresses in depth because of tamper momentum is similar to Bousinesq solution. Poran and Rodriguez (1992) also did model tests for studying dynamic compaction; they used a box of 1.22x1.22x1.22m3 filled with Boston dry sand and prepared some design curves. Oshima and Takada (1994), conducted some centrifuge model tests, they studied effect of dynamic compaction operational parameters, like: tamper weight, drop height and tamper radius for estimation of the improvement depth.
Jafarzadeh F. (2006) performed some model tests, he used a box with dimensions of 45x35x40 cm$^3$ and measured vertical stress and acceleration time history, Figure 2.17 shows vertical stress time histories for 1 kgf tamper dropped from 1m height.

![Diagram of transducers arrangement](image)

Figure 2.16 Transducers arrangement, adapted from Jafarzadeh F. (2006)
Thilakasiri and Gunaratne (1996), performed some experimental tests to determine induced dynamic impact stress in soft soils. In Figure 2.18 you can see time-history of experimental impact stress.
Oshima A. and Takada N. (1994) performed some centrifugal model tests for heavy tamping; they concluded that Impact duration $t_f$ is irrespective of $H$ and the initial slope of the acceleration vs. time coincide; Figure 2.19 and The impact duration $t_f$ is approximately proportional to the ram mass of unit base area $m/A$, irrespective of $H$. Figure 2.20 in addition, The relationship bet. Max. accel. And the square root of drop height shows linearity.
Figure 2.19 Ram acceleration time-history. Adapted from Oshima A. et al (1994)

Figure 2.20 Impact duration vs. ram mass of unit base area. Adapted from Oshima A. et al (1994)
In the context of the finite element method, DC impacts on the soil surface could be modeled via the following approaches:

(i) Applying a force time-history derived from impact acceleration records of tamper to the centroid of tamper; tamper modeled as a shell structural element.

(ii) Modeling real impact, by considering tamper as a shell structural element and defining shear and normal parameters of interface for modeling impact between tamper and soil surface by initializing free fall velocity to tamper grid points (nodes) as initial condition of problem.

(iii) Applying a pressure time-history, a half sinusoidal function, directly to the nodes, located beneath tamper zone without modeling tamper and interface.

(iv) Applying a normal velocity time-history, a half sinusoidal function, directly to the nodes located beneath tamper zone without modeling tamper and interface our
preliminary runs with modeling real impact show that settlement profile for cohesive soil (clay) is uniform - it is indentation not crater. Moreover, contact pressure is redistributed so that its magnitude at the center of tamper is more than that at the edge of tamper.

In this research, methods (ii), (iii), and (iv) are employed for modeling impact, but the obtained results show that the most accurate method is method (iv) - applying a half sinusoidal soil particle velocity directly to the nodes located beneath tamper zone. Because modeling real impact with considering tamper as a shell structural element and interface for modeling impact increase run time. In addition, it can cause some singularity at the edge of tamper and increase complexity of problem, because the need for modeling structural elements and interface. Eq. 2-23 is applied for converting normal stress to normal soil particle velocity as follows:

\[ \sigma_n = 2(\rho C_p) v_n, \]  

(2-23)

in which \( C_p = \sqrt{\frac{K + 4G/3}{\rho}} \). \( \sigma_n, \rho, C_p, v_n, K, \) and \( G \) are the applied normal stress, mass density, speed of p-wave propagation through medium, input normal particle velocity, bulk modulus, and shear modulus, respectively. According to soil properties and tamper operational parameters, the peak of normal particle velocity changes between 5 m/sec to 9 m/sec for cohesive soil. Throughout this research, a half sinusoidal time history of normal particle velocity with magnitudes of 5 to 7 m/sec is applied to the nodes located beneath tamper to simulate tamper impact.
2.17 Review of previous studies on DC simulation

Since 1970s when dynamic compaction (DC) was presented as an applicable method for soil remediation, many scholars have studied the behavior of soil under dynamic loads. To provide design methods that decrease the cost and time needed in trial compactions for setting-up the DC projects, the influence of important parameters on the results of DC were studied using various analytical, numerical, and laboratorial methods. In the Table 2.5 some of analytical and numerical studies performed by previous researchers has been summarized.
<table>
<thead>
<tr>
<th>Reference</th>
<th>Model Dim.</th>
<th>Type of Model</th>
<th>Summary</th>
</tr>
</thead>
<tbody>
<tr>
<td>Miller &amp; Pursey 1956</td>
<td>1-D</td>
<td>Elastic</td>
<td>$\sigma = \frac{SV}{\pi a^2 \omega} e^{-\left(\frac{R}{2M}\right)^2} \cos(\omega t - \cos^{-1} \frac{R \omega}{S})$</td>
</tr>
<tr>
<td>Kolsky; Lysmer 1963</td>
<td>1-D</td>
<td>Elastic</td>
<td>Calculate initial stress in soil surface $\sigma_0 = \rho c V$ c:dilation velocity</td>
</tr>
<tr>
<td>Scott &amp; Pearce</td>
<td>1-D</td>
<td>Elasto-Plastic unsaturated soil</td>
<td>Calculate vertical stress at surface vs. time</td>
</tr>
<tr>
<td>Roesset et al</td>
<td>1-D</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Chow et al (1992)</td>
<td>1-D</td>
<td>nonlinear</td>
<td>Predict crater depth, DI and present characteristic design curves for selecting: grid spacing</td>
</tr>
<tr>
<td>Chow et al (1992)</td>
<td>1-D</td>
<td>Linear</td>
<td>Deceleration of tamper</td>
</tr>
<tr>
<td>Thilakasiri</td>
<td>1-D</td>
<td>Non-linear</td>
<td></td>
</tr>
<tr>
<td>Lee et al</td>
<td>1-D</td>
<td>1-D wave propagation equations in pile during hammering</td>
<td>Similarity bet. Pile driving and heavy tamping</td>
</tr>
<tr>
<td>Gunaratne et al</td>
<td>1-D</td>
<td>Experimental</td>
<td>Estimate: EPWP in saturated soil,</td>
</tr>
<tr>
<td>Poran &amp; Rodriguez</td>
<td>2-D</td>
<td>Elasto-Plastic moel with large deformation formulation</td>
<td>They concluded that mesh refinement and reassignment of soil parameters should be considered during impact</td>
</tr>
<tr>
<td>Pan &amp; Selby</td>
<td>2-D</td>
<td>Total stress analysis, non-associated Mohr–Coulomb plasticity model</td>
<td></td>
</tr>
<tr>
<td>Gu &amp; Lee</td>
<td>2-D</td>
<td>Total stress Analysis, Elasto-Plastic Cap Model</td>
<td>Discuss: effects of drop energy, momentum of the falling tamper and tamper radius on the depth of improvement</td>
</tr>
<tr>
<td>Ghasseimi &amp; Pak</td>
<td>2-D</td>
<td>Coupled Hydro-Mechanical code Drucker-Prager</td>
<td>PWP response during impact assessed and they concluded that most of the DC improvement occurs during the undrained phase</td>
</tr>
<tr>
<td>Querol L.(2008)</td>
<td>2-D</td>
<td>Pastor–Zienkiewicz constitutive model in coupled code formulation</td>
<td>PWP response presented, a wave of dryness could be noticed</td>
</tr>
</tbody>
</table>
2.18 Vacuum consolidation

Vacuum consolidation method is a ground improvement technique that by applying negative vacuum pressure to an isolated soil mass decreases the water pore pressure inside soil. Therefore, via the reduction of the water pore pressure in the soil the effective stress enhances without changing the total stress. The improvement of construction methods and the development of designing technique have resulted in that the vacuum consolidation method is become an effective technique for soft soil improvement and capable to perform in various soil types. Other application such as expediting dewatering and consolidation of leachate materials are also examined by engineers. Similar fundamentals as those have been used in surcharge preloading by vertical drains are applied in the vacuum consolidation method too; in fact, the vacuum pressure surcharge is used to consolidate soft saturated soils at the same manner of embankment surcharge principles. The surcharge is exerted by a vacuum load which has the order of 50 to 80 kPa that is the same as a 4m embankment of material with density of 1500 kg/m3.

2.19 History of Vacuum Consolidation

In practice, Kjellman (1952) proposed the use of vacuum pressure as a surcharge for consolidating soft cohesive soil. Later, the above-mentioned idea was employed by the Royal Swedish Geotechnical Institute as a technique for the soft soil ground improvement. The basis of the technique is to decrease the atmospheric pressure of soil via the application of negative vacuum pressure to the isolated soil mass, which leads to
the increase of the effective stress without changing the total stress by decreasing the pore water pressure in the soil.

Various problems that were mainly in keeping constant of effective vacuum pressure during treatment forbade the wide application of this method for several decades. However, the improvement of this technique and the production of better vertical drains and vacuum pumps made vacuum consolidation method applicable in many projects. The other factors including the clarification of the principles of vacuum preloading, the improvement of construction equipment and the development of new design techniques resulted in the effective and affordable application of this method. Also, the above-mentioned factors made it suitable in various projects whether on-land or underwater.

2.20 Principles of vacuum preloading

As Indraratna et al. (2007) mentioned, Some issues like long consolidation time when applying surcharge loading alone, unsafe height of embankment for providing
required loading, or when affording embankment fill material isn’t affordable; make it necessary the application of other methods in combination with surcharge loading. One of the practical techniques is the applying vacuum pressure inside soil mass. Via the applying negative pore water pressure, without changing the total stress a higher effective stress is reached, propagation of the vacuum surcharge via vertical drains causes inward movement of soil around vacuum pipes, whereas the embankment surcharge preloading results in the outward deflection of soil mass. Therefore, by combining vacuum and surcharge preloading the outward lateral displacements reduce, and hence, the risk of damage to neighbor structures and abrupt undrained failure decrease. Figure 2.23 compares lateral displacement and stress-path for vacuum and surcharge preloading, in addition Figure 2.24 shows that for an equivalent loading condition both of surcharge preloading and vacuum preloading produce the same vertical effective stress inside soil mass.
Figure 2.23 (a) later expansion for surcharge preloading and lateral contraction and tension cracks for vacuum preloading (b) comparison of stress-path for vacuum and surcharge preloading. Adapted from Indraratna, B. et al (2007)
Instrumentations

For monitoring performance and quality of vacuum application, various instrumentations are essential in any vacuum consolidation project. Some of soil responses to vacuum application like settlement and lateral displacement and change of pore water pressure should be measured in addition discharged water and vacuum pressure should be monitored too, during a vacuum consolidation project. With these data
the geotechnical engineer can estimate degree of consolidation and judge stability of embankment.

Figure 2.25 Instrumentation placement for vacuum consolidation. Adapted from Indraratna et al. (2005)

2.22 Vacuum consolidation design

For designing a vacuum preloading project Terzaghi 1-D consolidation theory can be applied practically. Loan T. K. et al. (2006) explained design parameters as: individual block area, drain spacing, depth, effective vacuum pressure, and vacuum pumping period and soil properties, in the following sections these designing parameters will be explained with more details.

2.22.1 Treatment area by single vacuum pump

Due to the size of the area under treatment, the vacuum consolidation project is divided into independent and isolated blocks, as Loan T. K. et al. (2006) explained that
size of block in Chinese standard generally changes in the range of 6,000 to 10,000m²; In Chinese practice it is considered that standard area for each single vacuum pump to be 1,000-1,500 m²; In the case of Menard vacuum consolidation method, the typical area for each vacuum pump is 5000-7000m². Note that in Japan the standard area equipped by a single vacuum pump is from 2,000 to 2500m², Loan T. K. et al. (2006)

2.22.2 Drain spacing

The spacing between vertical drains should be specified based on the horizontal coefficient of consolidation, $c_{hv}$, of the soil and the required consolidation time. When there is a balance between time and cost, this spacing would in general be optimized. The spacing between vertical drains generally lies in the range of 0.7m to 1.8m.

2.22.3 Effective depth

The drain length is specified based on the soil deposit stratigraphy and the required influence depth. Loan T. K. et al. (2006) mentioned that the treatment depth in the most projects of China and Japan was about 20m.

2.22.4 Vacuum pressure

The vacuum pressure within the soil also known as under-sheet vacuum pressure is generally smaller than the vacuum applied in the pump. The efficiency of the vacuum loading of 70-80% is considered in most projects. It should be mentioned that designed vacuum pressure in Menard vacuum consolidation system is 75 kPa. Loan T. K. et al. (2006) explained that In China, a value of 80 kPa is utilized as effective vacuum pressure, which results in greater values of efficiency, 80-95% and In Japan, a minimum value of 60 kPa is recommended to utilize for design.
2.22.5 Soil properties

The soil properties which are the most influencing ones in design of vacuum consolidation are the index of compression, $C_c$ and the horizontal coefficients of consolidation, $c_h$. Loan T. K. et al. (2006). These parameters can be found from Oedometer test conducted on the undisturbed samples in the laboratory.

2.22.6 Degree of consolidation and vacuum pumping duration

Loan T. K. et al. (2006) mentioned that In general to reach a suggested degree of consolidation; the vacuum pumping duration is obtained based on calculated final settlement through Terzaghi 1-D consolidation theory. It is noteworthy to mention that the ratio of observed settlement to computed final settlement from Terzaghi theory is selected as the degree of the consolidation at the time in many vacuum consolidation projects.

2.23 Important parameters influencing vertical drain behavior

Indraratna et al (2007) explained factors affecting vertical drains efficiency as follow: (1) vertical drain equivalent diameter, (2) apparent opening size of drain, (3) discharge capacity (4) smear zone, and (5) drain unsaturation. In following sections each factors explained by details

2.23.1 Equivalent diameter of vertical drain

For designing purposes usually it is assumed that water flows into a drain with circular cross-section. Hence, for the rectangular cross section of the PVD it is necessary to convert them to an equivalent circular one. To this end, the following equations are
usually used to convert a rectangular band drain with width \( a \) and thickness \( b \) to an equivalent circular drain with diameter \( d_w \),

\[
\begin{align*}
d_w &= 2\left(\frac{a + b}{\pi}\right) \quad \text{(Hansbo, 1981)} \\
d_w &= \left(\frac{a + b}{2}\right) \quad \text{(Atkinson and Eldred, 1981)}
\end{align*}
\]

(2-24) (2-25)

2.23.2 Apparent opening size of PVD filter

Filter of PVD should prevent the entrance of the soil particles to the opening for avoiding clogging and at the same time water should pass through the drain, therefore PVD filter should satisfy these two fundamental but contrasting requirements. A common relation for the permeability of the drain is as below

\[
K_{\text{filter}} > 2K_{\text{soil}}.
\]

(2-26)

The intrusion of soil particles through the filter can be minimized using an effective filtration (Carroll, 1983). A commonly relation used as filtration requirement is as following:

\[
\frac{O_{95}}{D_{85}} \leq 2 - 3,
\]

(2-27)

Where, \( O_{95} \) and \( D_{85} \) are the approximate largest particle that would effectively pass through the filter and the diameter of clay particles corresponding to 85% passing, respectively.

2.23.3 Discharge capacity

As Indraratna et al (2007) mentioned the effective factors on the discharge capacity of the vertical drain are its cross section, filter permeability, and surrounding soil condition. Two important factors pertinent to the discharge capacity of a vertical drain are
consisting of (1) the estimation of the required discharge capacity in design and (2) the quantity of drain discharge capacity. As a result, the discharge capacity would be related to the following issues, the volume of drain channel pores, the lateral earth pressure applied to the vertical drain Figure 2.26, Indraratna et al (2007) explained that possible vertical drain deflections like folding, bending and twisting due to large vertical or horizontal settlement of soil, clogging of filter openings due to entrance of fine soil particles, as well as the corrosion of PVD materials. Based on the above-mentioned factors, the required discharge capacity, \( q_{\text{req}} \), is presented as (Chu et al., 2004):

\[
q_{\text{req}} \geq 7.85F_s k_h l_m^2
\]  

(2-28)

in which \( F_s = 4-6; \) \( l_m \) and \( k_h \) are maximum discharged length and soil permeability, respectively.

Figure 2.26 discharge capacity vs lateral pressure (adopted from Indraratna et al (2007) )
2.23.4 Smear zone

The installation of vertical drains into soft ground using a steel mandrel results in the remolding of the soil around mandrel, therefore rate of horizontal consolidation and permeability in this zone decrease in comparison with undisturbed soil around this zone. Eriksson et al. (2000) showed that the extent of smear zone related to shape and size of mandrel, method of pushing mandrel, and the type of soil. The diameter of the smear zone, \( d_s \), is related to the cross sectional area of mandrel via the following relation presented by Jamiolkowski et al. (1981)

\[
d_s = \frac{(5 \text{ to } 6) \ d_m}{2}
\]  

(2-29)

In which, \( d_m \) is the diameter of the circle that has the same area of mandrel. Another relation was presented by Hansbo (1981)

\[
d_s = 2d_m.
\]  

(2-30)

Indraratna (1997) using large-scale laboratory tests related \( d_s \) to the equivalent diameter of the drain as below

\[
d_s = (3 \text{ to } 4) d_w
\]  

(2-31)

Indraratna and Redana (1998) showed that the permeability near the drain is similar to the vertical permeability which remains almost without change. They exhibited that the ratio of horizontal to vertical permeability \((k_h/k_v)\) reaches to one close to the drain surface.
Both the variation of permeability and that of water content along the distance from mandrel can be used to assess the area of smear zone (Indraratna, 2006). The changes of the ratio of the horizontal to vertical permeability pertinent to various consolidation pressures, obtained from large-scale laboratory consolidation, along the distance from mandrel are presented in Figure 2.28. The change of water content with respect to the radial distance is presented in Figure 2.29. The water content reduces by approaching toward drain and increases by approaching toward the bottom of cell at all radial locations (Indraratna, 2006).
Figure 2.28  $k_h/k_v$ along the distance from the mandrel (adapted from Indraratna and Redana 1995)

Figure 2.29 water content, and (b) normalized water content reduction with radial distance at a depth of 0.5 m (adapted from Indraratna, 2006)
2.23.5 Influence zone of drains

As illustrated in Figure 2.30, square or triangular patterns are applied to install the vertical drains. The area where affected by a single drain is called as the influence zone. For triangular and square grid spacing the influence radius, \( r_e \) is related to the drain spacing, \( S_p \) via the following relations.

\[
\begin{align*}
    r_e &= 0.564S_p \quad \text{(Square Pattern)} \\
    r_e &= 0.525S_p \quad \text{(Triangular Pattern)}
\end{align*}
\]  

\( r_e = \frac{S_p}{\sqrt{\pi}} = 0.564S_p \)  \quad \text{Square pattern} \\
\( r_e = S_p\sqrt{\frac{\sqrt{3}}{2\pi}} = 0.525S_p \)  \quad \text{Triangular pattern}

Figure 2.30 different grid spacing pattern for vertical drains (Walker, 2006)

2.24 Comparison of surcharge and vacuum preloading

Leong et al (2000) compared surcharge and vacuum preloading by laboratory experiments they concluded that Both the vacuum preloading and surcharge have similar output under the preloading pressure of 80 kPa, but vacuum consolidation has the following two benefits including no requirement for fill material and having shorter treatment time. When there is no leakage problem and the water table is near the surface
the use of vacuum preloading would be interesting. The device which is used by Leong et al (2000) for conducting laboratory experiments illustrated in Figure 2.31, this apparatus was utilized for applying vacuum pressure to soil mass.

![Figure 2.31](image)

**Figure 2.31 Pressure plate apparatus for applying vacuum pressure (adapted from Leong et al 2000)**

Using laboratory experiments performed on a soft soil deposit, Leong et al (2000) concluded that for an equivalent loading condition, shear strength improvement achieved by surcharge preloading is greater than that produced by vacuum preloading. The experimental results for stress condition similar to vacuum preloading illustrate that the maximum shear strength is obtained at a matric suction of smaller than 100 kPa. And after that increasing matric suction results in decreasing shear strength. The reduction of shear strength is related to the desaturation of the soil.
Figure 2.32 Comparison of water content change for vacuum and surcharge preloading (adapted from Leong et al 2000)

Figure 2.33 Comparison of strength gain for vacuum and surcharge preloading (adapted from Leong et al 2000)
2.25 Vacuum Consolidation Theory

Mohamedelhassan and Shang (2002) used Terzaghi’s consolidation theory, for explaining the average degree of consolidation for combined vacuum and surcharge preloading.

\[ U_{vc} = 1 - \sum_{m=0}^{\infty} \frac{2}{M^2} \exp(-M^2 T_{vc}) \]

(2-34)

\[ M = \frac{\pi}{2} (2m + 1) \]

(2-35)

\[ T_{vc} = c_{vc} t / H^2 \]

(2-36)

Where:

- \( T_{vc} \): time factor for combined vacuum and surcharge preloading
- \( c_{vc} \): coefficient of consolidation for combined vacuum and surcharge preloading
- \( H \): depth of clay layer
- \( t \): consolidation time

Indraratna et al. (2004) explained that vacuum distribution along vertical drain isn’t uniform and it decrease with increasing depth, in addition Indraratna et al suggested a radial consolidation theory based on laboratory experiments for non-uniform trapezoidal distribution of suction pressure, Figure 2.34. Calculated average excess pore pressure ratio \( (R_u = \Delta p / \bar{u}_0) \) for radial consolidation with considering vacuum preloading calculated by:
Figure 2.34 Vacuum pressure distribution in the vertical drain (adopted from Indraratna et al. 2005)

\[ R_u = \left(1 + \frac{p_0 (1+k_1)}{u_0} \right) \exp \left( -\frac{\theta T_h}{\mu} \right) - \frac{p_0 (1+k_1)}{u_0} \] (2-37)

And

\[ \mu = \ln \left( \frac{n}{s} \right) + \left( \frac{k_h}{k_s} \right) \ln(s) - 0.75 + \pi z (2l - z) \frac{k_h}{q_w} \left\{ 1 - \frac{k_h - 1}{\frac{k_h}{k_s} (\frac{n}{s})} \right\} \] (2-38)

Where

- \( P_0 \): vacuum at the top of vertical drain
- \( k_1 \): ratio between the vacuum at the top and bottom of the drain
- \( \bar{u}_0 \): the in-situ EPWP
- \( k_h \): the horizontal permeability coefficient of soil in the smear zone
- \( T_h \): the time factor
- \( n \): the ratio \( d_c/d_w \) (\( d_c \) is the diameter of the equivalent soil cylinder and \( d_w \) is the diameter of the drain)
- \( s \): ratio \( d_s/d_w \) (\( d_s \) is diameter of smear zone)
$z =$ depth

$l =$ the equivalent length of drain

$q_w =$ the well discharge capacity
CHAPTER III

DYNAMIC COMPACTION & VACUUM CONSOLIDATION NUMERICAL SIMULATION CONSIDERATIONS

3.1 Introduction

Understanding soil behavior under impact loads has been an interesting subject for researchers. Many analytical, numerical and experimental studies have been carried out to predict the important parameters involved in DC treatments, but in the literature there isn’t a comprehensive numerical, analytical and experimental study for D.C. on saturated/partially saturated cohesive soil, considering effect of pore water pressure on soil responses during dynamic compaction.

In this chapter, modeling techniques for coupled hydro-mechanical simulation of dynamic compaction have been presented. Two cap constitutive model, namely Cap-Yield and Modified Cam-Clay constitutive models were used to capture the highly non-linear behavior of soil. In addition, initial in-situ stresses and pore water pressure initialized inside soil domain prior to start of dynamic and consolidation analysis were considered.

The objective of finite difference simulation (FD) is investigating the effect of dynamic compaction operational parameters, soil properties, and initial degree of saturation, on soil responses, like: generation of excess pore water pressure, change of mean effective stress, lateral and vertical displacement, crater depth and etc. In addition,
some transient flow analysis was conducted for modeling vacuum consolidation after
generation of non-uniform excess pore water pressure during dynamic impacts for
assessing surface settlement.

This chapter covers the development of FD model, including element type,
 element size, dynamic time step, model domain, boundary conditions, material damping,
methodology for simulating unsaturated conditions, and modeling impact.

3.2 Introduction to Flac3D

FLAC3D is a three-dimensional explicit finite-difference program for simulating
soil and rock structures. Simulation domain constructed by polyhedral elements within a
three-dimensional node. Behavior of each element controlled by linear or nonlinear
stress/strain law in response to applied load and boundary conditions. As mentioned in
Flac3D manual, the explicit, Lagrangian calculation scheme and the mixed-discretization
zoning technique used in this software, for accurately modeling of plastic collapse and
flow. In Flac3D help, one of the software benefits explained as automatic inertia scaling
and automatic damping which will not affect the mode of failure.

3.3 Advantages of Flac3D in comparison with Other Methods

Here, it is summarized some of the Flac3D advantages in comparison with other
methods:

1. The “mixed discretization” scheme (Marti and Cundall 1982) applied in Flac3D, for
precise simulation of plastic collapse loads and plastic flow.

2. In Flac3D, full dynamic equations of motion are calculated for both dynamic and static
simulations; therefore Flac3D could follow physically unstable processes without
numerical distress, (Flac3D manual).
3. An “explicit” solution method is applied in Flac3D, which significantly will reduce run time of nonlinear problems.

4. FLAC3D is capable of handling any constitutive model with no adjustment to the solution algorithm, but in most finite element codes different solution methods are needed for different constitutive models.

3.4 Description of 1-D finite difference model

1-D DC is simulated in a soil column with a height of 30m using Cap-Yield constitutive model. 1-D finite difference mesh including dimensions and boundary conditions is shown in Figure 3.1. The loading is defined by Eqs. (3-1) and (3-2). The maximum pressure stresses, $P_{\text{max}}$ applied at the top of column are equal to 1.5e6Pa, 1.0e6Pa, 0.5e6Pa and 0.2e6Pa, respectively. In the following, the boundary conditions considered in the present research are explained; at the bottom, normal quiet boundary is applied for avoiding reflection of outward waves. In the lateral boundaries, horizontal displacements are kept fixed. Analysis is undrained, and pore water pressure at surface of column is set to zero due to free draining boundary. Clay is considered as normally consolidated, and the preconsolidation pressure is set to initial in-situ stresses. The size of elements is 0.3m, and dynamic time step is 1e-6sec.

$$P(t) = P_{\text{max}} \sin(10\pi t), \quad \text{if } t \leq 0.1 \text{ sec}, \quad (3-1)$$

$$P(t) = 0, \quad \text{if } t > 0.1 \text{ sec}. \quad (3-2)$$
3.5 Element Type, Mesh Size and Dynamic Time Steps

Table 3.1 summarizes modeling approach used in this research. Element type used is brick. Mesh size or element size should be carefully selected for the problem of wave propagation in a semi-infinite half space. Very small element size can cause numerical instability according to a study by Zerwer et al. (2002). On the other hand, large elements cannot allow the shorter waves to travel, which are associated with highest frequencies. Vailliappan et al. (1984) indicated the minimum size to allow the propagation of short wavelengths. The time increment is selected, so that the primary waves, which are the fastest, can be recorded on two consequent nodes along the travel distance. Below equations provide mathematic expressions of the requirements.
\[ g \leq \zeta \lambda_{\text{min}} = \zeta \frac{V_s}{f_{\text{max}}} \quad (3-3) \]

\[ T \leq \frac{g}{V_p} \quad (3-4) \]

Where:

- \( g \): element size
- \( V_s \): secondary wave velocity
- \( V_p \): primary wave velocity
- \( f_{\text{max}} \): maximum input frequency
- \( \zeta \): equals 0.25 for mass matrices that are consistent and equals 0.2 for lumped masses
- \( T \): minimum sampling time

In this research with Flac3D, Dynamic time step should be less than critical dynamic time step calculated by equation (3-5) and Mesh size should be less than equation (3-6)

\[ \Delta t_{\text{crit}} = \min \left\{ \frac{V}{c_p A_{\text{max}}} \right\} \quad (3-5) \]

Where:

- \( C_p \): is the \( p \)-wave speed
- \( V \): is the tetrahedral subzone volume
- \( A_{\text{max}}^{f} \): is the maximum face area associated with the tetrahedral subzone

\[ \Delta l \leq \frac{\lambda}{10} \quad (3-6) \]

Where:

- \( \lambda \) is the wavelength associated with the highest frequency component that contains appreciable energy.
Table 3.1 Simulation Items

<table>
<thead>
<tr>
<th>Item</th>
<th>This Research</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model Dimensions</td>
<td>10x10x20m</td>
<td>Pan and Selby: 5x10^3 sec for force–time loading approach, and 1x10^-6 s for rigid body impact</td>
</tr>
<tr>
<td>Dynamic Time Step</td>
<td>1x10^-6 s</td>
<td></td>
</tr>
<tr>
<td>Soil Mesh Type</td>
<td>Brick</td>
<td></td>
</tr>
<tr>
<td>Soil Mesh Size</td>
<td>0.25m</td>
<td>Assuming Rayleigh wave propagation velocity=30m/sec and maximum frequency=10 Hz minimum element size $g \leq \phi_{\min} = \frac{V_g}{f_{\max}} = 0.25 \times \frac{29}{10} = 0.725$</td>
</tr>
<tr>
<td>Soil Constitutive Model</td>
<td>Soil constitutive model is Cap-Yield Implemented in Flac3D</td>
<td></td>
</tr>
<tr>
<td>Tamper Element</td>
<td>Tamper elements are liner shell structure with high elastic modulus</td>
<td>Tamper fall freely on soil surface and real impact modeled</td>
</tr>
<tr>
<td>Interface</td>
<td>Interface constitutive model is Mohr-Coulomb with normal and shear stiffness, $k_n=8e7N/m^3$, $k_s=8e4 N/m^3$, $\Phi=10$ and $c=0$ and sliding in interface is allowed</td>
<td></td>
</tr>
<tr>
<td>Boundary Condition</td>
<td>Vertical Boundaries are infinite and Bottom boundary is viscous</td>
<td></td>
</tr>
</tbody>
</table>

3.6 Description of 3-D Finite Difference Model

Throughout this section, a 3-D model is employed to assess the pore water pressure response of soil mass during impact. Note that the used mesh is consisting of polyhedral elements. Mesh size or element size should be carefully selected for the problem of wave propagation in a semi-infinite half space. The time increment is
selected, so that the primary waves, which are the fastest, can be recorded on two consequent nodes along the travel distance. In this research using Flac3D, dynamic time step should be less than critical dynamic time step calculated by Eq. (3-7) and mesh size should be less than that given by Eq. (3-8).

\[ \Delta t_{\text{crit}} = \min \left\{ \frac{V}{C_p A_{\text{max}}^f} \right\}, \quad (3-7) \]

\[ \Delta l \leq \frac{\lambda}{10}. \quad (3-8) \]

Where, \( C_p \), \( V \), and \( A_{\text{max}}^f \) are the p-wave speed, tetrahedral subzone volume, and the maximum face area associated with the tetrahedral subzone, respectively. \( \lambda \) is the wavelength associated with the highest frequency component that contains appreciable energy.

Pan and Selby (2002) suggested the use of dynamic time step of 1e-6sec for the rigid body impact modeling. 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 0.25m 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 Figure 3.2 and Figure 3.3, respectively.
Figure 3.2 Top view of 3-D finite difference mesh with dimensions and boundary conditions.

Figure 3.3 Elevation view of 3-D finite difference mesh.
Boundary conditions are as follows. At the bottom, the normal, strike, and dip quiet boundary, also called damper or absorbent boundary, is applied for avoiding reflection of outward waves. Lateral boundary conditions are infinite.

3.7 Mechanical Damping and Material Response

DC on soft soil, clay, majority of energy dissipated by plastic flow and deformation therefore type of damping does not have significant effect on the results. In this research, Rayleigh damping was used; it was originally applied in the analysis of structures and elastic continua, to damp the natural oscillation modes of the system. The equations, therefore, are expressed in matrix form. A damping matrix, $C$, is used, with components proportional to the mass ($M$) and stiffness ($K$) matrices:

$$C = \alpha M + \beta K$$  \hspace{1cm} (3-9)

Where: $\alpha=\text{the mass-proportional damping constant}$; and $\beta=\text{the stiffness-proportional damping constant}$. In this simulation, damping constants were selected as $\alpha=2$ and $\beta=20$ as recommended by Flac3D help for cohesive soils.

3.8 Modeling unsaturated condition

In FLAC3D’s formulation, pore pressure is set to zero if the saturation at any node is less than 1, (Flac3D manual). The effect of dissolved and trapped air may be allowed by reducing the local fluid modulus while keeping the saturation at 1, it is supposed that there is an equivalent fluid present throughout the pore space. Although no pore pressures are present in a partially saturated region, the trapped fluid still has weight, and the fluid moves under the action of gravity at a reduced apparent permeability, (Flac3D manual). The initial saturation may be given by the user, but it is also updated
during FLAC3D’s calculation cycle as necessary to preserve the mass balance. Note that in FLAC3D, saturation is not considered as an independent variable; it cannot be fixed at any grid point. For groundwater problems, the bulk modulus of water may be different in different parts of the grid; to account for the varying amounts of air present.

Menard and Broise (1975) considered compressibility of saturated soil due to the presence of micro-bubbles as one of the fundamentals of DC. Natural water inside soil deposits contains entrapped air pockets in the voids between soil particles, and these micro-bubbles increase compressibility of air-water mixtures, and therefore decrease generation of excess pore water pressure during impact considerably. For example, Chaney (1978) mentioned that the water bulk modulus can be decreased by an order of magnitude for an air-water mixture at 99% saturation in compacted sand. Hence, it is essential to take into account the compressibility of air-water mixture instead of using the compressibility of pure water. Yoshimi et al. (1989) performed some element tests, cyclic torsional shear tests on hollow cylindrical specimens, to study the effect of initial degree of saturation on the underained cyclic shear strength (liquefaction resistance) of air-pluviated Toyoura sand. The tests conducted by Yoshimi et al. (1989) are in good agreement with the theoretical relationship proposed by Lade and Hernandez.

\[
B = \frac{\frac{1}{1 + nK_s} \left[ \frac{S_r}{K_w} + \frac{(1-S_r)}{u_a} \right]}{(3-10)}
\]

where \(n\), \(K_s\), \(K_w\) and \(u_a\) are porosity, bulk modulus of soil skeleton, bulk modulus of pure water \((2.23 \times 10^6 \text{ kPa})\), and absolute pressure in pore fluid, respectively. In this research, Eq. (3-11) is used for finding equivalent bulk modulus of water, \(K'_w\)

\[
\frac{1}{K'_w} = \frac{S_r}{K_w} + \frac{1-S_r}{u_a}. \quad (3-11)
\]
How Eq.(3-11) is obtained from Eq.(3-10) is explained in the following paragraph. For cohesive fine grained soils, since permeability is low and impact time is small too, we can suppose that analysis is undrained during impact, and water content, \( \omega \) is constant during undrained phase. So, from Eq. (3-12) it is found that \((S, e)\) is constant during impact. Regarding the initial soil conditions, \( S_r, e, \) and pore water pressure are known. Consequently, we can calculate \( S_r, e \) degree of saturation in each step from Eq. (3-12), and \( K'_w \) will be recalculated for every step from Eq. (3-11).

\[ S_r e = \omega G_s \]  

(3-12)

Note that, \( e \) and \( G_s \) in Eq. (3-12) are void ratio and specific gravity, respectively.

To simulate unsaturated condition in Flac3D, it is supposed that we have a saturated soil, \( S_r = 100\% \), with an equivalent bulk modulus, reduced bulk modulus, that has the same B-Value of an unsaturated soil. In unsaturated soil, we have three phases, trapped air, pure water, and soil skeleton. For saturated sample by considering \( S_r=1.0 \) and water bulk modulus equal to reduced one, \( K'_w \), B-value, \( B_1 \) calculated from Eq. (3-13); for unsaturated sample B-value, \( B_2 \) calculated from Eq. (3-14) after equalizing \( B_1 \) and \( B_2 \) Eq.(3-11) obtained.

\[ B_1 = \frac{1}{1+nK_s \left[ \frac{1}{K'_w} \left( \frac{1-S_r}{u_a} \right) \right]} = \frac{1}{1+nK_s \left[ \frac{1}{K'_w} \right]} \]  

(3-13)

\( B_2 \) is calculated from Eq. (16) as below

\[ B_2 = \frac{1}{1+nK_s \left[ \frac{1-S_r}{K'_w} \frac{1}{u_a} \right]} \]  

(3-14)

\[ B_1 = B_2 \Rightarrow \frac{1}{K'_w} = \frac{S_r}{K'_w} + \frac{1-S_r}{u_a} \]  

(3-15)
Eq. (3-15) is employed to find the equivalent bulk modulus of water, $K_w'$ at the end of each step.

3.9 Modeling Impact

In the context of the FE/FD methods, DC impacts could be modeled via the following approaches:

(i) Applying a force time-history derived from impact acceleration records of tamper to the centroid of tamper; tamper is modeled as a shell structural element.

(ii) Modeling real impact, by considering tamper as a shell structural element and defining shear and normal parameters of interface for modeling impact between tamper and soil surface by initializing free fall velocity to tamper grid points (nodes) as initial condition of problem.

(iii) Applying a pressure time-history, a half sinusoidal function, directly to the nodes, located beneath tamper zone without modeling tamper and interface.

(iv) Applying a normal velocity time-history, a half sinusoidal function, directly to the nodes located beneath tamper zone without modeling tamper and interface.

Our preliminary runs with modeling real impact show that settlement profile for cohesive soil (clay) is uniform - it is indentation not crater. Moreover, contact pressure is redistributed so that its magnitude at the center of tamper is more than that at the edge of tamper. Settlement profile to model a real impact of rigid tamper with soil surface is shown in Figure 3.4.
In this research, methods (ii), (iii), and (iv) are employed for modeling impact, but the obtained results show that the most accurate method is method (iv) - applying a half sinusoidal soil particle velocity directly to the nodes located beneath tamper zone. Because modeling real impact with considering tamper as a shell structural element and interface for modeling impact increase run time. In addition, it can cause some singularity at the edge of tamper and increase complexity of problem, due to the need for modeling structural elements and interface.

Eq. (3-17) is applied for converting normal stress to normal soil particle velocity as follows:

$$\sigma_n = 2\left(\rho C_p\right) v_n,$$

(3-17)

In which $C_p = \sqrt{\frac{K + 4G/3}{\rho}}$, $\sigma_n$, $\rho$, $C_p$, $v_n$, $K$, and $G$ are the applied normal stress, mass density, speed of p-wave propagation through medium, input normal particle velocity, bulk modulus, and shear modulus, respectively. According to soil properties and tamper operational parameters, the peak of normal particle velocity changes between 5 m/sec to 9 m/sec for cohesive soil. Throughout this research, a half sinusoidal time history of normal particle velocity with magnitudes of 5 to 7 m/sec is applied to the nodes.
located beneath tamper to simulate tamper impact; and for some simulations real impact was modeled too.

3.10 Mechanical constitutive models:

As Ghassemi et al. (2010) mentioned Cap models have been successfully used for simulation of DC in soils by Thilakasiri (2001), Gu & Lee (2002). These models have been extended over the years to incorporate isotropic or kinematic hardening when the need arose. In FLAC3D there are 3 cap constitutive models for modeling soil behavior:

3.10.1 Modified Cam-Clay

The modified Cam-clay model is a hardening/softening elastoplastic constitutive model. This model is nonlinear and mean effective stress correlated to volumetric strain for modeling hardening/softening nature of soil. The failure envelopes have the shape of ellipsoids which rotated about the mean effective stress, in addition the shear flow rule is associated; elements with this constitutive model couldn’t tolerate tensile mean effective stress (Flac3D manual). In this model change of mean effective stress correlated to elastic volumetric strain according to Eq.(3-18) and tangent bulk modulus calculated from Eq.(3-19).

\[ \Delta p = \frac{v p}{\kappa} \Delta \varepsilon_p^e \]  

\[ K = \frac{v p}{\kappa} \]  

Where:

p: mean effective pressure

v: specific volume

\( \kappa \): swelling line slope
\( \varepsilon_p^e \): plastic volumetric strain

\( K \): bulk modulus

3.10.1.1 Yield Function

The yield function for every particular value \( p_c \) of the pre-consolidation pressure has the form, (Flac3D manual)

\[
f(q,p) = q^2 + M^2 p(p - p_c)
\]

(3-20)

Where

\( M \): is a material constant, slope of critical state line,

\( p_c \): pre-consolidation pressure

\( q \): deviatoric stress

\( p \): mean effective pressure

3.10.1.2 Hardening/Softening rule

The size of the yield curve is dependent on the value of the pre-consolidation pressure, \( p_c \). Eq.(3-21) shows cap hardening rule for Modified Cam-Clay

\[
\frac{dp_c}{d\varepsilon_p} = \frac{p_c}{\lambda - \kappa} \ast (\nu_\lambda - \lambda \ast \ln \left( \frac{p_c}{p_{ref}} \right))
\]

(3-21)

It has 4 parameters \( \lambda, \kappa, \nu_\lambda \) and \( p_{ref} \) where \( \lambda \): compression line slope, \( \kappa \): swelling line slope, \( \nu_\lambda \): reference specific volume and \( p_{ref} \): reference pressure

3.10.1.3 Cam-Clay BaseLine Parameters

The baseline of Cam-clay parameters used in parametric study for preparing design procedure for HVDM are summarized in Table 3.2 and they are representative of a normally consolidated clay with un-drained shear strength of 7kPa at depth of 2m
Table 3.2 Modified Cam-Clay parameters

<table>
<thead>
<tr>
<th>M</th>
<th>λ</th>
<th>k</th>
<th>Reference Pressure(kPa)</th>
<th>Reference Specific Volume((v_\lambda))</th>
<th>Density((\rho)) Kg/m3</th>
<th>K₀</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>0.1</td>
<td>0.04</td>
<td>1</td>
<td>2.5</td>
<td>1500</td>
<td>0.6</td>
</tr>
</tbody>
</table>

It has 5 parameters M, λ, κ, Reference Pressure and Reference Specific Volume

3.10.2 Double-Yield

Plastic volumetric strain caused by the application of isotropic pressure considered in this model, in this model similar to Modified Cam-Clay the cap surface defined by cap pressure and its hardening behavior is activated by volumetric plastic strain (Flac3D manual) and follows a piecewise-linear law which defined by user through table. The tangential bulk and shear moduli evolve as plastic volumetric strain progress and they are defined as a factor, R, which defined as the ratio of elastic bulk modulus to plastic bulk modulus.

3.10.3 Cap-Yield

Cap-Yield (Cy) model has frictional strain-hardening and softening behavior, an elliptic volumetric cap with strain-hardening behavior, and an elastic modulus function of plastic volumetric strain. This model is capable of more realistic representation of the loading/unloading response of soils. The Cy-soil model is a strain-hardening constitutive model characterized by a frictional Mohr-Coulomb shear envelope and an elliptic volumetric cap with ratio of axes, defined by a shape parameter, \(\alpha\). Cy-soil model has three types of hardening laws: a cap hardening law, for considering the volumetric power law behavior observed in isotropic compaction; a friction-hardening law, to simulate the hyperbolic stress-strain law behavior observed in drained triaxial tests; and a compaction/dilation law to model irrecoverable volumetric strain taking place as a result
of soil shearing, (Flac3D manual). Because in hydro-mechanical coupled modeling of
dynamic compaction in saturated or partially saturated soft soils, high excess pore water
pressure would be generated and effective mean stress would become negative in some
elements; therefore during impact, Modified Cam-Clay couldn’t be used as it is not able
to support tensile stresses or negative mean effective stresses, consequently in parametric
study for deriving predictive equation for generation of EPWP, Cap-Yield constitutive
model was used. In Figure 3.5 Cam-Clay and Cap-Yield yield criterions are compared
together.

Figure 3.5 red one is Cap-Yield and blue one is Cam-Clay yield criterion in q-p’-
plane for both of them pc is 100kPa and α=0.66 and M=1.0
Cap-Yield constitutive model parameters summarized in Table 3.3

Table 3.3 Cap-Yield Constitutive model parameters

<table>
<thead>
<tr>
<th>Item</th>
<th>Property Name</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>alpha</td>
<td>cap yielding surface parameter, $\alpha$ yield surface is represented by an ellipse in (q,p)-plane, and $\alpha$ is equal to vertical diagonal divided by horizontal diagonal of ellipse</td>
</tr>
<tr>
<td>2</td>
<td>bulk</td>
<td>maximum elastic bulk modulus, $K$</td>
</tr>
<tr>
<td>3</td>
<td>cap_pressure</td>
<td>Current consolidation pressure, $p_c$</td>
</tr>
<tr>
<td>4</td>
<td>cptable</td>
<td>number of table relating cap pressure ($p_c$) to plastic volume strain ($ev_{plastic}$)</td>
</tr>
<tr>
<td>5</td>
<td>density</td>
<td>mass density, $\rho$</td>
</tr>
<tr>
<td>6</td>
<td>dilation</td>
<td>ultimate dilation angle, $\psi$</td>
</tr>
<tr>
<td>7</td>
<td>dilation_mob</td>
<td>mobilized dilation angle, $\psi_m$</td>
</tr>
<tr>
<td>8</td>
<td>dtable</td>
<td>number of table relating mobilized dilation angle to plastic shear strain</td>
</tr>
<tr>
<td>9</td>
<td>friction</td>
<td>ultimate friction angle, $\phi$</td>
</tr>
<tr>
<td>10</td>
<td>friction_mob</td>
<td>mobilized friction angle, $\phi_m$</td>
</tr>
<tr>
<td>11</td>
<td>ftable</td>
<td>number of table relating mobilized friction angle to plastic shear strain</td>
</tr>
<tr>
<td>12</td>
<td>multiplier</td>
<td>multiplier on current plastic cap modulus to give elastic bulk and shear moduli, $R$</td>
</tr>
<tr>
<td>13</td>
<td>shear</td>
<td>maximum elastic shear modulus, $G$</td>
</tr>
</tbody>
</table>

3.10.3.1 Yield and Potential Functions

In Cap-Yield constitutive model, Shear yield criterion and flow rule are defined by a Mohr Coulomb criterion. The yield envelope explained in Eq. (3-22), (Flac3D Manual):

$$f = Mp' - q,$$  \hspace{1cm} (3-22)

where, $p'$ is the mean effective stress, $p' = -(\sigma'_1 + \sigma'_2 + \sigma'_3)/3$, and $q$ is a measure of shear stress defined as below:

$$q = -[\sigma'_1 + (\delta - 1)\sigma'_2 - \delta\sigma'_3]$$  \hspace{1cm} (3-23)

$\sigma'_1$, $\sigma'_2$ and $\sigma'_3$ are effective principal stresses, and $\delta$ and $M$ are defined as follows:
\[ \delta = \frac{(3 + \sin \varphi_m)}{(3 - \sin \varphi_m)}, \quad (3-24) \]

and

\[ M = \frac{6\sin \varphi_m}{(3 - \sin \varphi_m)}. \quad (3-25) \]

where, \( \varphi_m \) is the mobilized friction angle. The potential function is non-associated and has the following form:

\[ g = M^* p' - q^*, \quad (3-26) \]

in which the following relations are satisfied,

\[ q^* = [\sigma_1' + (\delta^* - 1) \sigma_2' - \delta^* \sigma_3'], \quad (3-27) \]

\[ \delta^* = \frac{(3 + \sin \psi_m)}{(3 - \sin \psi_m)}, \quad (3-28) \]

and

\[ M^* = \frac{6\sin \psi_m}{(3 - \sin \psi_m)} \quad (3-29) \]

3.10.3.2 Volumetric Cap Criterion and Flow Rule

Flow rule is associated, and the yielding criterion defined as (Flac3D Manual):

\[ f_c = q^2 \frac{\alpha^2}{\alpha^2} + p'^2 - p_c^2. \quad (3-30) \]

in which, \( \alpha \), is a dimensionless parameter, defines the shape of the elliptical cap in the (\( p' - q \)) plane, and \( p_c \) is the cap pressure. The hardening curve relating the cap pressure, \( p_c \) to the cap plastic volumetric strain, \( e^p \) is generated by means of a user-defined table. If there is no table, \( p_c \) is assumed to be constant and equal to the input value of the cap pressure, (Flac3D manual).
3.10.3.3 Cy-Soil cap hardening

Soil stiffness usually increases nonlinearly with increasing confining pressure. In Cy-Soil mode a cap hardening table is used for specifying a power law behavior. Cap hardening rule of Cap-Yield model is (Flac3D Manual):

\[
\frac{dp_c}{de} = \frac{1+R}{R} K_{ref}^{iso} \left( \frac{p'}{p_{ref}} \right)^m.
\]  

(3-31)

There are four parameters in Eq. (3-31) consisting of \( K_{ref}^{iso}, p_{ref}, m \) and \( R \). Note that the elastic bulk modulus is determined as (Flac3D Manual):

\[
K^e = (1 + R) K_{ref}^{iso} \left( \frac{p'}{p_{ref}} \right)^m.
\]  

(3-32)

Here, \( K_{ref}^{iso} \) is the slope of the laboratory curve for \( p' \) versus \( e \), volumetric strain, at reference effective pressure, \( p_{ref} \). Furthermore, \( m \) is a constant with the condition \( m<1 \), and \( R \) is a constant equal to the ratio of the elastic modulus to the hardening modulus (Flac3D Manual).

3.11 Fluid constitutive model

In this research fl_iso fluid constitutive model that is isotropic fluid flow model was used for drained and un-drained analyses and its parameters are as below:

3.11.1 Permeability, \( k \):

This parameter is independent and is not updated by Flac3D during calculations but it can be coupled with all mechanical or dynamic parameters, such as, plastic volumetric strain, fluid time, dynamic time, effective stress, etc. for considering change of permeability.
3.11.2 Void ratio or porosity

Porosity is used by FLAC$^{3D}$ to calculate the saturated density of the medium and evaluate Biot modulus in the case when the fluid bulk modulus is given as an input.

3.11.3 Biot coefficient and Biot Modulus or Fluid Bulk Modulus

For considering calculations for compressible grains, Biot coefficient is used and defined as below (Flac3D Manual):

$$\alpha = 1 - \frac{K}{K_s}$$

(3-33)

$K$: drained bulk modulus

$K_s$: bulk modulus of solid component

In this research, it is supposed that soil grains are incompressible; therefore, $\alpha$ is equal to 1

Biot modulus is defined as:

$$M = \frac{K_{ud} - K}{\alpha^2}$$

(3-34)

$K_{ud}$: undrained bulk modulus of the soil

For an incompressible solid constituent ($\alpha = 1$)

$$M = \frac{K_s}{\eta}$$

(3-35)

Where $\eta$ is the porosity.

3.12 Summary and Conclusions

In this chapter, FD models have developed to simulate coupled hydro-mechanical dynamic compaction in partially saturated cohesive soil. Soil domain was modeled via polyhedral elements. For avoiding reflection of outward waves, free field boundary was used at the edges and the normal, strike, and dip quiet boundaries were used at the
bottom. Initial in-situ stresses were initialized to be consistent with soil dry density, initial degree of saturation, and porosity inside soil mass.

For modeling impact between tamper and soil surface two methods were used in the present study. (1) Modeling real impact, by considering tamper as a shell structural element and defining shear and normal parameters of interface for modeling impact between tamper and soil surface by initializing free fall velocity to tamper grid points (nodes) as initial condition of problem. (2) Applying a normal velocity time-history, a half sinusoidal function, directly to the nodes located beneath tamper zone without modeling tamper and interface. In addition, for the comparison purposes, our numerical model results are compared with those of Querol et al. (2008). They used a two-dimensional plane strain u-w model,

The following summarizes the contribution of the presented work compared to previous works available in literature:

- Initial state of soil was modeled by initializing horizontal and vertical stresses inside soil domain.
- 2 cap models: Modified Cam-Clay and Cap-Yield constitutive models were used to describe soil behavior in which the highly non-linear stress-strain behavior of soils could be captured
- A numerical simulation technique was presented considering the change of water bulk modulus with degree of saturation Sr with particular emphasis on excess pore pressure generation during compaction
- Mohr-Coulomb interface was used for simulating real impact between tamper and soil surface.
• Coupled hydro-mechanical simulation performed in this research by coupling generation of excess pore water pressure to volumetric strain
CHAPTER IV
EXCESS PORE WATER PRESSURE GENERATION FOR PARTIALLY SATURATED CLAY DURING DYNAMIC COMPACTION

4.1 Introduction

Consideration and techniques for developing numerical modeling of DC in saturated and partially saturated cohesive soils were presented in Chapter III. Flac3D with Cap-Yield constitutive model was used to investigate the influence of dynamic compaction operational parameters and initial degree of saturation on generation of EPWP. Once numerical model was developed, it can be used to achieve a better understanding of the behavior of cohesive soils under DC, during which fluid and mechanical properties are coupled together. The present work considers the influence of degree of saturation on the responses of the soft soil (clay) and develops useful working/prediction equations for generation of excess pore water pressure (EPWP) due to DC. In this chapter an extensive parametric study was performed for investigating the influence of initial degree of saturation and operational parameters of DC on the distribution and magnitude of generated EPWP due to undrained DC.

4.2 One-dimensional (1-D) model results for DC

As mentioned in Chapter III, 1-D DC model was developed with height of 30m using Cap-Yield constitutive model. Soil properties for this simulation are summarized in
Table 4.1, and the loading is defined by Eqs. (3-1) and (3-2) and the maximum pressure stresses, $P_{\text{max}}$, applied at the top of column are equal to $1.5 \times 10^6 \text{Pa}$, $1.0 \times 10^6 \text{Pa}$, $0.5 \times 10^6 \text{Pa}$ and $0.2 \times 10^6 \text{Pa}$, respectively.

<table>
<thead>
<tr>
<th>$K_i (\text{Pa})$</th>
<th>$P_{\text{ref}} (\text{Pa})$</th>
<th>$m$</th>
<th>$R$</th>
<th>$\alpha$</th>
<th>$\text{Max Bulk (Pa)}$</th>
<th>Friction</th>
</tr>
</thead>
<tbody>
<tr>
<td>$2.5 \times 10^7$</td>
<td>$1 \times 10^5$</td>
<td>0.5</td>
<td>0.66</td>
<td>1.0</td>
<td>$1 \times 10^5$</td>
<td>20</td>
</tr>
</tbody>
</table>

The computed results for maximum dynamic stress of $1.5 \times 10^6 \text{ Pa}$ are shown in Figure 4.1 in which negative EPWP is generated at superficial depths. It is noteworthy to mention that beneath the applied pressure in all of the examined degree of saturation, EPWP increase linearly with depth until reaching to maximum value, and beyond the maximum point EPWP decreases. The maximum value changes from 20 kPa for initial degree of saturation 85% to 80 kPa for initial degree of saturation 99%. Furthermore, depth of maximum EPWP increases with increase of initial degree of saturation. For initial degree of saturation of 85%, the depth of maximum residual EPWP is approximately 2.0 m, and for initial degree of saturation of 99%, it is 8 m. Considering the generated EPWP as a measure for assessing depth of maximum influence, it can be concluded that this depth increases with initial degree of saturation. When initial degree of saturation is 100%, EPWP increases linearly with depth that is in agreement with Querol (2008) results for 1-D model, using Pastor-Zinkovich and elastic constitutive models with constant water bulk modulus of $1 \times 10^9 \text{Pa}$, showed EPWP generation increased with depth.
Figure 4.1 Residual EPWP versus depth for different initial degree of saturations for $P_0 = 1.5e6$ Pa.

Figure 4.2 shows the peak of EPWP versus depth for different initial degree of saturations. As it can be seen the peak of EPWP decreases with depth. For initial degree of saturation of 99%, all of applied $P_0$, 1.5e6 Pa, converted to EPWP, at the surface.
Figure 4.2 Peak of EPWP for different saturations for $P_0$ 1.5e6 Pa.

Figure 4.3 shows the surface settlement of 1-D column versus initial degree of saturation. From this figure it is understood that settlement decreases with increasing initial degree of saturation. On one hand, this observation is due to the fact that increasing initial degree of saturation results in more voids filled with water, which results in not having any void for settlement. On the other hand, increasing initial degree of saturation would generates more negative EPWP at superficial depths, and thus the effective stress would increase, resulting in less settlement.
Figure 4.3 Surface settlement versus initial degree of saturation for $P_0 \ 1.5e6 \ Pa$.

Figure 4.4 exhibit the relationship between EPWP versus time generated at depth 0.5m below surface for different initial degree of saturations. It is seen that EPWP increases abruptly to a high level, and drops to residual EPWP after a short time. Moreover, it is found that the increase of degree of saturation increases the maximum EPWP; however, the residual EPWP remains approximately the same for all initial degree of saturations. As it can be seen in Figure 4.4, a high negative residual EPWP is generated near the surface for initial degree of saturation at 100%.
Figure 4.4EPWP time-history for different initial degree of saturations at depth 0.5m below surface for $P_0$ 1.5e6 Pa.

Figure 4.5 shows the change of water bulk modulus versus time at depth 1m for initial degree of saturations at 85%, 90% and 92%, respectively. During applying compression pressure wave, water bulk modulus increases to its maximum value and after that it drops down to residual water bulk modulus. Though this rise of water bulk modulus plays an important role on soil and pore water pressure responses, this fact has been ignored in all of the previous researches.
Figure 4.5 Change of water bulk modulus for different initial saturation of 90% at depth 1m.

Figure 4.6 shows change of degree of saturation due to DC for different initial degree of saturation. It can be seen that close to the surface this change is negative, but with increasing depth, it increases until reaching maximum and decrease, afterwards. If one compares Figure 4.1 with Figure 4.6, it can be seen that depth of maximum change of saturation and maximum EPWP coincide for different initial degree of saturation. This means that the depth that has maximum change of saturation also has the maximum EPWP. It is obvious that EPWP and degree of saturation develop in the same manner.
4.3 Parametric study for EPWP predictive equation in 1-D simulation

An extensive parametric study using Flac3D code is carried out to examine the effects of influencing factors: initial degree of saturation and magnitude of dynamic stress on top of soil column, on EPWP generation in normally consolidated cohesive soils. The goal of this study is considering effect of initial degree of saturation on generation of EPWP, when P-wave propagates through a 1-D soil column. To predict residual and peak EPWP, three effective parameters including dynamic stress ($P_0$), initial degree of saturation ($S_r$) and depth ($z$) are considered.

Metaheuristic techniques have been introduced to solve highly nonlinear and complex modeling and optimization problems. Artificial neural network have (ANN) been known as the most widely used metaheuristic modeling tools for engineering problems. A main constraint of ANN is that it is usually trapped in local minima. Therefore and to reach an optimal solution, an ANN should be initially trained using a
metaheuristic global optimization algorithm. Ledesma et al. (2009) have recently hybridize ANN and developed a well-known global optimization algorithm named simulated annealing (SA) to cope this issue and improve the ANN efficiency.

Here, the hybrid ANN and SA, called SA-ANN is utilized to derive the relationships of residual and peak EPWP. This method has been successfully proposed by Alavi and Gandomi (2011) for complex geotechnical engineering problems. The employed hybrid system uses the SA strategy to assign good starting values to the weights of the network before performing optimization. The SA-ANN algorithm is implemented by using the Neural Lab Software for ANNs.

To empirically drive the formula, a database is created containing 1680 datasets. The range of dynamic stress and initial degree of saturation in this parametric study is summarized in Table 4.2. It should be noted that the depth is varied between 0 to 17.7 m.

Table 4.2 1-D Parametric study parameters.

<table>
<thead>
<tr>
<th>Initial degree of saturation, Sr (%)</th>
<th>Magnitude of dynamic stress, $P_0$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>85</td>
<td>0.2</td>
</tr>
<tr>
<td>90</td>
<td>0.5</td>
</tr>
<tr>
<td>92</td>
<td>1.0</td>
</tr>
<tr>
<td>95</td>
<td>1.5</td>
</tr>
<tr>
<td>97</td>
<td>2.5</td>
</tr>
<tr>
<td>99</td>
<td></td>
</tr>
<tr>
<td>100</td>
<td></td>
</tr>
</tbody>
</table>

Out of the whole datasets 85% was used for training, and the rest of them for testing the general capability of the model. The final formula for prediction both residual and peak EPWP simultaneously is:

$$\begin{align}
\{ \text{residual} & \} = \text{Logsig} \left( \text{Bias} + \sum_{h=1}^{n_h} V_h \times \text{Logsig} \left( \text{bias}_h + \sum_{i=1}^{n_i} w_{ih} x_i \right) \right) \\
\{ \text{peak} & \} = \text{Logsig} \left( \text{Bias} + \sum_{h=1}^{n_h} V_h \times \text{Logsig} \left( \text{bias}_h + \sum_{i=1}^{n_i} w_{ih} x_i \right) \right)
\end{align}$$ (4-1)
Where $nh$ is the number hidden neurons which is equal to 4, $ni$ is the number of inputs which is equal to 3, and $\text{logsig}$ is of form $1/(1 + e^{-x})$. It should be noted that all the inputs and outputs are linearly normalized between 0.1 and 0.9 to be useful in the ANN model. The weights and biases used in Eq. 4-1 are presented in Table 4.3 and Table 4.4.

Table 4.3 Weight ($W_{ih}$) and bias values between the input and hidden layer

<table>
<thead>
<tr>
<th>Inputs</th>
<th>Number of hidden neurons ($h$)</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>$x_{1i} (P_{0,n})$</td>
<td>3.5350</td>
<td>2.0867</td>
<td>-3.4541</td>
<td>2.6155</td>
<td></td>
</tr>
<tr>
<td>$x_{2i} (S_{rn})$</td>
<td>14.4238</td>
<td>2.0984</td>
<td>-14.4386</td>
<td>2.6149</td>
<td></td>
</tr>
<tr>
<td>$x_{3i} (Z_{n})$</td>
<td>-7.4067</td>
<td>-8.4167</td>
<td>6.3983</td>
<td>-17.3139</td>
<td></td>
</tr>
<tr>
<td>bias$_{h}$</td>
<td>-13.7039</td>
<td>-2.7999</td>
<td>13.8127</td>
<td>-1.8099</td>
<td></td>
</tr>
</tbody>
</table>

* $P_{0,n}$, $S_{rn}$ and $Z_{n}$ are the normalized values.

Table 4.4 Weight ($V_{hi}$) and bias values between the hidden and output layer.

<table>
<thead>
<tr>
<th>Output</th>
<th>Number of hidden neurons ($h$)</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Residual</td>
<td>-35.6552</td>
<td>15.5804</td>
<td>-34.9735</td>
<td>-9.6916</td>
<td>33.9100</td>
</tr>
<tr>
<td>Peak</td>
<td>-3.9349</td>
<td>1.2187</td>
<td>-6.9439</td>
<td>0.5923</td>
<td>4.7532</td>
</tr>
</tbody>
</table>

Table 4.5 Statistical results of the developed model.

<table>
<thead>
<tr>
<th>Actual vs Predicted</th>
<th>Actual / Predicted</th>
</tr>
</thead>
<tbody>
<tr>
<td>$^1$RMSE (%)</td>
<td>$^2$MAE (%)</td>
</tr>
<tr>
<td>Residual</td>
<td>6.3637</td>
</tr>
<tr>
<td>Peak</td>
<td>6.1538</td>
</tr>
</tbody>
</table>

$^1$RMSE is Root Mean Square Error  

$^2$MAE is relative Mean Absolute Error  

$^3$R$^2$ is Coefficient of Determination  

$^4$Std Dev is Standard Deviation  

$^5$COV is Coefficient of Variation

The statistical results of the proposed model are presented in Table 4.5. As it can be seen all the values presented in this table indicate the performance accuracy of the SA-ANN model.
An illustrative example is provided here to further explain how Eq. (4-1) can be used for calculating residual and peak EPWP in 1-D P-wave propagation through a soil column.

For this propose, a sample is taken. The $P_0$, $Sr$ and $z$ values for this sample are, respectively, equal to 0.5 Mpa, 99%, and 2.4 m. The calculation has three main steps as follow:

i) Normalization of the input data,

Linearly normalization of the input data ($P_0$, $Sr$ and $z$) to lie in a range from 0.1 to 0.9 as:

For $P_0 = 0.5$ Mpa with the maximum and minimum values of 0.2 and 1.5 the formulation is

$$P_{n0} = 0.1 + 0.8 \left( \frac{P_0 - 0.2}{1.5 - 0.2} \right) = 0.284615$$  \hspace{1cm} (4-2)

For $Sr = 99\%$ with the maximum and minimum values of the variable are 85 and 99, thus:

$$Sr_{n} = 0.1 + 0.8 \left( \frac{Sr - 85}{99 - 85} \right) = 0.9$$  \hspace{1cm} (4-3)

For $z = 2.4$ m with the maximum and minimum values of 0 and 17.7 the formulation is

$$z_{n} = 0.1 + 0.8 \left( \frac{z}{17.7} \right) = 0.208475$$  \hspace{1cm} (4-4)

ii) Calculation of the normalized residual and peak EPWP ($\text{residual}_{n}$ and $\text{peak}_{n}$) using the weights between different layers

$$\text{residual}_{n} = \text{Logsig} \left( 33.91 - 35.6552F_1 + 15.5804F_2 - 34.9735F_3 - 9.6916F_4 \right) = 0.5569$$  \hspace{1cm} (4-5)

$$\text{peak}_{n} = \text{Logsig} \left( 4.7532 - 3.9349F_1 + 1.2187F_2 - 6.9439F_3 + 0.5923F_4 \right) = 0.2272$$  \hspace{1cm} (4-6)

Where

$$F_i = \text{Logsig} \left( -13.7039 + 3.5350P_{n0} + 14.4238Sr_{n} - 7.4067z_{n} \right) = 0.2209$$  \hspace{1cm} (4-7)
\[ F_2 = \text{Logsig}(-2.7999 + 2.0867P_{0,n} + 2.0984\text{Sr}_n - 8.4167z_n) = 0.1118 \] (4-8)

\[ F_3 = \text{Logsig}(13.8127 - 3.4541P_{0,n} - 14.4386\text{Sr}_n + 6.3983z_n) = 0.7629 \] (4-9)

\[ F_4 = \text{Logsig}(-1.8099 + 2.6155P_{0,n} + 2.6149\text{Sr}_n - 17.3139z_n) = 0.0894 \] (4-10)

iii) Denormalize residual and peak EPWP to obtain the predicted values.

\[
\text{residual}_n = 0.1 + 0.8 \left( \frac{\text{residual} + 18288.06}{77914.52 + 18288.06} \right) \Rightarrow \text{residual} = 120253\text{residual}_n - 30313.4 = 36668 \text{ Pa}
\] (4-11)

\[
\text{peak}_n = 0.1 + 0.8 \left( \frac{\text{peak} - 277}{1496477 - 277} \right) \Rightarrow \text{peak} = 1870250\text{peak}_n - 186748 = 238173 \text{ Pa}
\] (4-12)

It should be noted that the actual residual and peak EPWP are 37154 (Pa) and 239248 (Pa), respectively.

4.4 Three-dimensional (3-D) model results for DC

As mentioned in chapter III, a 3-D model was developed to assess the pore water pressure response of soil mass during impact; EPWP generation during DC for 3-D domain has been discussed in this section. In Figure 4.7 to Figure 4.11, the EPWP within the analyzed geometry calculated via the Cap-Yield constitutive model is given for different initial degree of saturations. Soil parameters of Cap-Yield for 3-D model are the same as 1-D simulation, presented in Table 4.1 These figures illustrate the generated EPWP for vertical lines parallel to CL. It is concluded that for high degree of saturation, like 99% and 98%, close to the CL and below tamper, negative EPWP is generated and
generally the EPWP increases with depth until reaching to a maximum value, then EPWP decreases with depth.

Figure 4.7 Residual EPWP profile at CL for initialized crater depth of 0.318m and tamper rad. 1.0m

Figure 4.8 Residual EPWP profile at line 0.7m away from CL for initialized crater depth of 0.318m and tamper rad. 1.0m
Figure 4.9 presents EPWP generation for the vertical line at the edge of tamper. Inasmuch as we have high impact energy, a negative EPWP is produced close to the surface below the edge of tamper.

Figure 4.9 Residual EPWP profile at line 1.0m away from CL. for initialized crater depth of 0.318m and tamper rad. 1.0m

Figure 4.10 and Figure 4.11 show EPWP for vertical lines outside of the edge of tamper. As it can be seen in these figures, for all initial degree of saturation, EPWP is approximately close to zero at surface and increases until reaching to a maximum, and then, it decreases with depth.
Figure 4.10 Residual excess pore water pressure profile at line 2.46m away from CL. for initialized crater depth of 0.318m and tamper rad. 1.0m

Figure 4.11 Residual excess pore water pressure profile at line 4.23m away from CL. for initialized crater depth of 0.318m and tamper rad. 1.0m
Figure 4.12 to Figure 4.15 show the peak of EPWP at different depths. As it can be seen, close to the CL, the peak of EPWP decreases with depth; however, when we go far from CL, the peak of EPWP increases with depth, and reach to a maximum value, and then decreases with depth.

Figure 4.12 Peak of EPWP profile at CL. for initialized crater depth of 0.318m and tamper rad. 1.0m
Figure 4.13 Peak of EPWP profile at line 1.0m away from CL. for initialized crater depth of 0.318m and tamper rad. 1.0m

Figure 4.14 Peak of EPWP profile at line 2.11m away from CL. for initialized crater depth of 0.318m and tamper rad. 1.0m
Figure 4.15 Peak of EPWP profile at line 3.17m away from CL. for initialized crater
depth of 0.318m and tamper rad. 1.0m

Figure 4.16 to Figure 4.20 illustrate the generated residual EPWP at different
depths. At depths near surface, for example $z = 0.5\ m$, due to combination of such effects
as high energy of impact, negative EPWP wave, non-uniform distribution of impact
pressure, and unsaturation conditions, it is difficult to state for which initial degree of
saturation that the magnitude of EPWP is higher. However, it is clear that for the initial
degree of saturation at 99%, the generated EPWP is less than other ones. Furthermore, it
is observed that the greater initial degree of saturation results in more EPWP increase
with depth.
Figure 4.16 Residual EPWP at depth 0.5m

Figure 4.17 Residual EPWP at depth 1.0m
Figure 4.18 Residual EPWP at depth 2.0m

Figure 4.19 Residual EPWP at depth 4.0m
Figure 4.21 shows EPWP generated in soil mass for initial degree of saturation at 99%. Upon consideration of the variation of water bulk modulus during impact, when the initial degree of saturation is 99%, the water bulk modulus varies between $1e7$ and $5e7Pa$ depending upon the magnitude of pore water pressure. As it can be seen in Figure 4.21, when water bulk modulus is updated in each time step to be proportional to the pore water pressure and degree of saturation, then small negative residual EPWP is generated close to the surface. Note that when water bulk modulus is considered to be constant at $5e7Pa$, a considerable negative EPWP will be induced until depth 4m. It is concluded that to simulate partially saturated conditions, simply by reducing only water bulk modulus will not lead to reasonable results. In fact, water bulk modulus should be updated in each time step in accordance with the pore water pressure and degree of saturation.
4.5 Parametric study for developing EPWP predictive equation in 3-D domain.

For the 3D case the same SA-ANN is again utilized to derive the relationship between residual EPWP and four dimensionless parameters, including $r/R$, $Z/R$, $Cr/R$ and $Sr$ (%), where $r$ is distance from C.L., $z$ is depth, $Cr$ is crater depth, $Sr$ is initial degree of saturation and $R$ is tamper radius. A huge database including 20,950 datasets obtaining from FDM analysis was used here. The range of input parameters varied in this parametric study is summarized in Table 4.6.
Table 4.6 Parametric study parameters for 3-D domain

<table>
<thead>
<tr>
<th>Initial degree of saturation, Sr (%)</th>
<th>Radius of Tamper (m)</th>
<th>Crater Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>85</td>
<td>0.5</td>
<td>0.318</td>
</tr>
<tr>
<td>90</td>
<td>1.0</td>
<td>0.445</td>
</tr>
<tr>
<td>92</td>
<td>1.5</td>
<td>0.573</td>
</tr>
<tr>
<td>95</td>
<td>2.0</td>
<td></td>
</tr>
<tr>
<td>97</td>
<td>2.5</td>
<td></td>
</tr>
<tr>
<td>99</td>
<td></td>
<td></td>
</tr>
<tr>
<td>100</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Out of the whole datasets, 85% of date was used for training and the rest of data was used for testing the general capability of the model. The final formula for prediction of residual EPWP is:

\[
\text{Residual EPWP} = \logsig \left( \text{Bias} + \sum_{h=1}^{n_h} V_h \times \logsig \left( bias_h + \sum_{i=1}^{n_i} w_{ih} x_i \right) \right) \tag{4-13}
\]

Where \( n_h \) is the number hidden neurons which is equal to 6, \( n_i \) is the number of inputs which is equal to 4. It should be noted that all the inputs and outputs are linearly normalized between 0.1 and 0.9 to be useful in the ANN model. The weights and biases used in Eq. (4-13) are presented in Table 4.7 and Table 4.8.

Table 4.7 Weight \((W_{ih})\) and bias values between the input and hidden layer

<table>
<thead>
<tr>
<th>Inputs</th>
<th>Number of hidden neurons ((h))</th>
</tr>
</thead>
<tbody>
<tr>
<td>(x_1), ((r/R_n))</td>
<td>1</td>
</tr>
<tr>
<td>(x_2), ((Z/R_n))</td>
<td>-14.9963</td>
</tr>
<tr>
<td>(x_3), ((Cr/R_n))</td>
<td>0.5520</td>
</tr>
<tr>
<td>(x_4), ((Sr_n))</td>
<td>-10.5147</td>
</tr>
<tr>
<td>(bias_h)</td>
<td>-2.1161</td>
</tr>
</tbody>
</table>

Table 4.8 Weight \((V_h)\) and bias values between the hidden and output layer.

<table>
<thead>
<tr>
<th>Number of hidden neurons ((h))</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bias</td>
<td>-6.9282</td>
<td>9.4893</td>
<td>-3.2716</td>
<td>2.0819</td>
<td>-9.4717</td>
<td>0.4260</td>
</tr>
</tbody>
</table>

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Statistical results of the residual EPWP prediction using the proposed algorithm are presented in Table 4.9. From this table, it is clear that the SA-ANN model has very good performance and match with the actual results.

<table>
<thead>
<tr>
<th>Actual vs Predicted</th>
<th>Actual / Predicted</th>
</tr>
</thead>
<tbody>
<tr>
<td>RMSE (%)</td>
<td>RMAE (%)</td>
</tr>
<tr>
<td>2.3302</td>
<td>1.0621</td>
</tr>
</tbody>
</table>

An illustrative example is provided to further explain the implementation of the proposed SA-ANN residual EPWP stresses. For this propose, a sample used for the testing of the models is taken. The r/R, Z/R, Cr/R and Sr (%) values for this sample are, respectively, equal to 1.93875, 3.25, 0.2225 and %97. The calculation has three main steps as:

i) Normalization of the input data,

Linearly normalization of the input data (P₀, sr and z) to lie in a range from 0.1 to 0.9 as:

For \( r/R = 1.93875 \) with the maximum and minimum values of 0 and 16.92 the formulation is

\[
\left( \frac{r}{R} \right)_n = 0.1 + 0.8 \left( \frac{r/R}{16.92} \right) = 0.19167
\]

(4-14)

For \( Z/R = 3.25 \) with the maximum and minimum values of the variable are 0 and 24, thus:

\[
\left( \frac{Z}{R} \right)_n = 0.1 + 0.8 \left( \frac{Z/R}{24} \right) = 0.2083
\]

(4-15)

For \( Cr/R = 0.2225 \) with the maximum and minimum values of 0.1272 and 1.146 the formulation is

\[
\left( \frac{Cr}{R} \right)_n = 0.1 + 0.8 \left( \frac{Cr/R - 0.1272}{1.146 - 0.1272} \right) = 0.1748
\]

(4-16)
For $S_r = 97\%$ with the maximum and minimum values of the variable are 85 and 99, thus:

$$(Z/R)_n = 0.1 + 0.8 \left( \frac{Z/R - 85}{99 - 85} \right) = 0.7857 \quad (4-17)$$

ii) Calculation of the normalized residual EPWP ($\text{residual}_n$) using the weights between different layers

$$\text{residual}_n = \text{Logsig} \left( 1.9810 - 6.9282 F_1 + 9.4893 F_2 - 3.2716 F_3 + 2.0819 F_4 - 9.4717 F_5 + 0.426 F_6 \right) = 0.7147 \quad (4-18)$$

where

$$F_1 = \text{Logsig} \left( -2.1161 - 14.9963 (r/R)_n + 0.5520 (Z/R)_n - 10.5147 (Cr/R)_n + 2.1559 S_r \right) = 0.0066 \quad (4-19)$$

$$F_2 = \text{Logsig} \left( 2.0389 - 15.3312 (r/R)_n - 26.8343 (Z/R)_n - 1.1328 (Cr/R)_n + 3.38 S_r \right) = 0.0174 \quad (4-20)$$

$$F_3 = \text{Logsig} \left( 1.2663 + 7.145 (r/R)_n + 3.954 (Z/R)_n + 2.7358 (Cr/R)_n + 2.1089 S_r \right) = 0.9963 \quad (4-21)$$

$$F_4 = \text{Logsig} \left( 2.4552 - 0.4628 (r/R)_n + 2.2375 (Z/R)_n + 5.9773 (Cr/R)_n + 2.4555 S_r \right) = 0.9970 \quad (4-22)$$

$$F_5 = \text{Logsig} \left( 2.1393 - 13.6335 (r/R)_n - 35.2217 (Z/R)_n - 0.3986 (Cr/R)_n + 3.9236 S_r \right) = 0.0082 \quad (4-23)$$

$$F_6 = \text{Logsig} \left( -4.4346 - 12.1280 (r/R)_n - 17.5142 (Z/R)_n + 2.3456 (Cr/R)_n + 10.8338 S_r \right) = 0.1846 \quad (4-24)$$
iii) Denormalize residual EPWP to obtain the predicted values with the maximum and minimum values of -283.4259 kPa and 103.6157 kPa.

\[
residual_n = 0.1 + 0.8 \left( \frac{peak + 283.4259}{103.6157 + 283.4259} \right) \Rightarrow residual = 483.802 peak_n - 331.806 = 13.9634 \text{ kPa}
\]

(4-25)

It should be noted that the actual residual EPWP of this case is 16.029 (kPa).

4.6 Other aspects of F.D. results

Stress path q versus \( p' \) during D.C. is presented in Figure 4.22 at different depths. For the vertical line 0.35m away from CL, it is observed that when impact wave reaches to a point near the surface, q and \( p' \) increase, and stress state point moves upward on critical state line (CSL). After wave has passed, the stress point moves downward on the critical state line, and after being on critical state line for a while, it drops down on the right side. After reaching to the minimum point, it increases and reaches to the critical state line again and then goes downward. For points farther from impact area, stress state moves upward on the critical state line and after wave has passed, the stress point moves downward, and after being on the line for a while drops down on the right side of the line.
Figure 4.22 Stress path for different depths at line 0.35m away from CL.

Figure 4.23 to Figure 4.25 present induced residual mean effective stress versus elevation for different initial degrees of saturation. For different vertical lines, the depth of maximum induced residual effective stress located at depths 2m to 3m. Moreover, it is found that the effective stress decreases with increasing initial degree of saturation. Considering mean effective stress as a measure for assessing depth of maximum influence, this depth located between depths 1/3DI to 1/2DI, where DI is, depth of influence.
Figure 4.23 Induced residual mean effective stress at line 0.35m away from CL.

Figure 4.24 Induced residual mean effective stress at line 2.11m away from CL.
Figure 4.26 to Figure 4.29 show the induced residual mean effective stress at different depths. From Fig. 30 which is for depth 0.5m below the surface, it can be seen that for the initial degree of saturation at 99%, the mean effective stress is more than other ones. This observation is due to the fact that near the surface we have high amount of impact energy and for the initial degree of saturation at 99%, negative EPWP is induced close to the surface. Therefore, the mean effective stress for the case of degree of saturation at 99% is greater than that of other initial degree of saturation. Note that by increasing depth, the higher mean effective stress is obtained for the smaller initial degrees of saturation.
Figure 4.26 Induced residual mean effective stress at depth 0.5m.

Figure 4.27 Induced residual mean effective stress at depth 1.0m.
Figure 4.28 Induced residual mean effective stress at depth 2.0m.

Figure 4.29 Induced residual mean effective stress at depth 3.0m.

Figure 4.30 exhibits the time history of water bulk modulus for the initial degree of saturation at 90% at different depths close to the CL. When the impact wave reaches
to the point, the water bulk modulus increases to a maximum value. Then, after the wave has passed, it decreases to the residual water bulk modulus.

Figure 4.30 Change of water bulk modulus for saturation 90% at different depths.

Figure 4.31 shows changes of induced residual saturation for different initial degree of saturation at depth 1.0m. Maximum change of induced residual saturation is approximately 1% for the initial degree of saturation at 85%.
Figure 4.31 illustrates the distribution of impact pressure at different time steps for initial degree of saturation at 90%. It can be found that for different instances, the impact stress at the CL is more than that at the edge of tamper. Figure 4.33 shows the peak of impact pressure for different initial degree of saturation. Since soil stiffness at the CL is more than that at edges, the magnitude of impact pressure at the CL is more than that at the edges; in addition, for soils with initial degree of saturation at 99%, it has lowest value of impact pressure. This is due to the fact that as more excess pore water pressure is generated during impact, then effective stress becomes less.
Figure 4.32 Impact pressure profile at different times after impact for initial saturation 90%.

Figure 4.33 Peak of impact pressure profile for different saturations.
Figure 4.34 to Figure 4.37 give the distribution of residual volumetric strain at different depths. From Figure 4.34 which presents the residual volumetric strain at depth 0.5m, it is understood that for the initial degree of saturation at 99%, the residual volumetric strain is more than that for the other initial degree of saturation. Due to greater value of water bulk modulus for higher initial degree of saturation, soil beneath tamper liquefies for those at higher initial degree of saturation, resulting in mean effective stress being close to zero at the time that EPWP is at the peak. Therefore, higher initial degree of saturation leads to more plastic volumetric strains close to surface. But with increasing depth, the higher initial degree of saturation leads to the lower compression volumetric strain.

Figure 4.34 Residual volumetric strain at depth 0.5m.
Figure 4.35 Residual volumetric strain at depth 1.0m.

Figure 4.36 Residual volumetric strain at depth 2.0m.
Figure 4.37 Residual volumetric strain at depth 4.0m.

Figure 4.38 shows typical profile of the lateral displacement variation with depth for normally consolidated clay at different vertical lines parallel to CL. For vertical lines close to the CL, for instance 0.35m away from CL, the lateral displacement decreases with depth. However, for vertical lines farther from CL, the lateral deflection increases with depth and reaches to a maximum value and after that the lateral deflection diminishes with depth. Maximum of lateral displacement happened at the edge of tamper close to the surface, for this simulation it is 1.0e-1m at depth 0.5m.
Figure 4.38 Lateral deflection of soil for initial degree of saturation 90%.

Figure 4.39 shows typical shape of acceleration-time history of tamper for real impact simulation, with W=15ton and H=10m, on a normally consolidated clay. As it can be seen that maximum acceleration is 45g, where g is gravitational acceleration. Figure 4.40 shows average of vertical force-unbalance parameters for elements beneath tamper, this parameter is one of the Fish variables in Flac3D and is representative of Force applied from tamper to soil surface, as it can be seen maximum force is close to 3.0e6N.
Figure 4.39 acceleration time history of tamper with $W=15\text{ton}$ $H=10\text{m}$

Figure 4.40 dynamic force exerted to soil surface by tamper
4.7 Pore water generation with constant water bulk modulus during impact

In all of the previous coupled hydro-mechanical works to model EPWP generation, the water bulk modulus was considered to be constant during impact. Furthermore, pure water bulk modulus, 2e9Pa or reduced water bulk modulus, for different degree of saturation, was initialized in soil mass. Nevertheless, water bulk modulus did not change during simulation. Our results show that when water bulk modulus is constant, high negative excess pore pressure is generated beneath tamper and close to the surface. Figure 4.41 shows excess pore water generation time-history at depth 2.0m close to the CL. As it can be seen EPWP rises abruptly to a high value and drops to a residual negative pore pressure. When water bulk modulus changes during impact, the time-history of EPWP has the same shape. In other words, it increases to a high value and drops to zero or negative pressure. Then, it increase and the residual EPWP becomes positive.

Figure 4.41 EPWP generation at depth 3m when water bulk modulus is constant 5e7Pa.
It is suggested that for modeling dynamic compaction in partially saturated soil, if water bulk modulus considered constant during impact, it is recommended reduced to a value, between 1e6 Pa to 1.5e7Pa, because using high values of water bulk modulus, say 1e9Pa or higher, leads to generation of high negative EPWP close to surface.

4.8 Comparisons with others

For the comparison purposes, our numerical model results are compared with those of Querol et al. (2008). They used a two-dimensional plane strain u-w model, and the applied maximum total force was 20 MN to a loading area of square, sized 4m (2m in the symmetrical model). The constitutive model used by them was Pastor-Zinkovich, parameters applied in the work of Querol et al. (2008), are provided in Table 4.10 and Table 4.11.

Table 4.10 Data used by Querol et al. [4] in the two-dimensional analysis of DC (adopted from [4]).

<table>
<thead>
<tr>
<th>ρ_s (kg/m³)</th>
<th>ρ_w (kg/m³)</th>
<th>K_s (Pa)</th>
<th>K_w (Pa)</th>
<th>n</th>
<th>k'(m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2720</td>
<td>1000</td>
<td>1x10^20</td>
<td>1x10^9</td>
<td>0.363</td>
<td>2.1x10^-3</td>
</tr>
</tbody>
</table>

Table 4.11 Pastor-Zienkiewicz parameters for plain strain DC analysis (adopted from [4]).

<table>
<thead>
<tr>
<th>K_0 (kPa)</th>
<th>G_0 (kPa)</th>
<th>M_g</th>
<th>M_r</th>
<th>α_g=α_r</th>
<th>H_0</th>
<th>β_0</th>
<th>β_1</th>
<th>γ_{DM}</th>
<th>H_{u0} (kPa)</th>
<th>γ_U</th>
</tr>
</thead>
<tbody>
<tr>
<td>45.0x10^3</td>
<td>22.5x10^3</td>
<td>1.32</td>
<td>0.545</td>
<td>0.45</td>
<td>350</td>
<td>4.2</td>
<td>0.2</td>
<td>4</td>
<td>6.0x10^3</td>
<td>2</td>
</tr>
</tbody>
</table>

The computed residual EPWP at the time of 1.5 sec within the analyzed geometry is shown in Figure 4.42. It is seen that from surface to depth 4m below the surface, the negative residual EPWP is generated in the range of -2.5e5 Pa to -4.5e5 Pa.
In the Cy-soil constitutive model, cap hardening and friction hardening are utilized for simulating Pastor-Zinkovich constitutive parameters used by Querol et al. (2008) for sand layer. Table 4.12 summarizes Cy-soil parameters. These parameters are approximately representative of medium sand.

Table 4.12 Cy-soil parameters used for comparison

<table>
<thead>
<tr>
<th>$K_{\text{iso}}^\text{ref}$ (kPa)</th>
<th>$p_{\text{ref}}$ (kPa)</th>
<th>$\nu$</th>
<th>$m$</th>
<th>$R$</th>
<th>$\alpha$</th>
<th>$G_{\text{ref}}^e$ (kPa)</th>
<th>$\varphi_f$ (deg.)</th>
<th>$\psi_f$ (deg.)</th>
<th>$R_f$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>3e4</td>
<td>100</td>
<td>0.2</td>
<td>0.5</td>
<td>0.66</td>
<td>1</td>
<td>3.5e4</td>
<td>32</td>
<td>4</td>
<td>0.9</td>
<td>0.35</td>
</tr>
</tbody>
</table>

Querol et al. (2008) used the constant water bulk modulus of 1e9 Pa. Here, we considered sand layer with the initial degree of saturation at 99%. Figure 4.43 presents the generated residual EPWP at different vertical lines parallel to CL. It can be found that
the generated residual negative EPWP is in good agreement with results of Querol et al. (2008).

![Figure 4.43 Residual EPWP generated in simulation geometry with Cy-soil constitutive model.](image)

4.9 Conclusions

In this research, Cap-Yield constitutive model is implemented in Flac3D program to model the dynamic compaction problem in saturated and/or unsaturated cohesive soils. Formulation is based on the displacement of solid phase, and the generation of EPWP is coupled to the volumetric strain. At the first stage, the suitability of this model is assessed for 1-D problem. Subsequently, it is applied to the 3-D model. In the following, the obtained results are summarized.

- Saturations between 95% and 99% can reduce the water bulk modulus by 2 to 3 orders. Therefore, use of a constant pure water bulk modulus, 2e9Pa, in simulating DC treatment could result in model errors. To get better results, similar to the elastic and
plastic soil stiffness, the water bulk modulus should be updated in each time step during impact.

- In this research, the water bulk modulus is treated as a function of pore water pressure and degree of saturation. When the water bulk modulus is constant during impact, high negative EPWP is generated beneath tamper; however, when the water bulk modulus is allowed to change during impact, the generated negative EPWP is small.

- For vertical lines close to the CL of tamper, the residual EPWP increases with depth. After reaching to the maximum value, EPWP reduces with depth. Note that by moving away from the CL, EPWP reduces linearly with depth.

- In 1-D simulations, it was observed that only the maximum value of EPWP is a function of initial degree of saturation. The residual EPWP is approximately the same for different initial degree of saturation.
CHAPTER V

SURFACE SETTLEMENT DUE TO APPLICATION OF VACUUM CONSOLIDATION AFTER GENERATION OF NON-UNIFORM EXCESS PORE WATER PRESSURE DUE TO

5.1 Introduction

“High vacuum densification Method” is a new technique of soft soil ground improvement, combining use of vacuum consolidation and deep dynamic compaction, and in literature referred to as HVDM. In this chapter, dynamic compaction and vacuum consolidation was simulated numerically in a cylinder domain using Flac3D, for a normally consolidated clay. In addition, this study presents three well-known formulation techniques, namely regression analysis, genetic programming and artificial neural networks for investigating the influence of initial degree of saturation, operational parameters of D.C. and amplitude of applied vacuum on the soil surface settlement. For each method, one of the most promising variant is chosen and used, and their final models are compared in terms of simplicity of the models and accuracy of the results. The comprehensive experimental database used for the development of the correlations was established upon a series of tests conducted in this study. For verification, an external validation was also carried out for all models using several criteria presented in the
literature. Sensitivity analysis of predictor variables of the proposed models was calculated to determine the significance of each of the variables to the soil settlement.

In this research, several alternative data mining approaches have been developed for the underlying relationship between inputs and an output and the related formulation. The classical formulation technique is regression analysis. It is the most well-known method in this field and has been used in many problems. The main issue in the regression analysis is that you should pre-define the formulation structure which can be very difficult for a complex problem and especially when you do not have any view about the physical meaning of the variables. The possible interactions between parameters make this problem even more difficult. Convergence and consistency are some other issues in this problem, which is mainly related to the defined structures. There are two regression analysis methods, linear and non-linear, however non-linear regression analysis (NLR) is more useful for complex engineering problems.

Genetic programming (GP) is another alternative approach for formulation, which searches a program space instead of a data space. Many researchers have recently employed GP and its variants to find out any complex relationships between the experimental data and especially for geotechnical engineering problems (e.g., Alavi and Gandomi (2011), Gandomi and Alavi (2012)). Gene expression programming (GEP) Oltean & Dumitrescu, (2002) is a recent variant of GP and it evolves: computer programs with different shapes and sizes. GEP can be utilized as an efficient alternative to the GP (Ferreira 2001). There were several scientific efforts directed at applying GEP to formulation of civil engineering systems Gandomi et al. (2012).
Artificial neural network (ANN) is one of the most widely used pattern recognition methods for complex and nonlinear problems. There have been many researches with the specific objective of applying ANNs to the geotechnical engineering problems (e.g., Das 2013). Here, the prediction ability of one of the most widely used ANN architecture, namely multilayer perceptron, Cybenko, (1989) (MLP) is investigated. ANNs are generally considered as black-box models. To overcome this issue and obtain an explicit form, conventional calculation procedure is proposed based on the fixed connection weights and biases factors of the final model.

The goal of this chapter is developing numerical models of HVDM in saturated/partially saturated cohesive soil using FLAC 3D in order to achieve a better understanding of the behavior of cohesive soils under dynamic compaction and vacuum consolidation. In this study, real impact of tamper on soil surface was modeled and after generation of non-uniform excess pore water pressure inside soil mass dynamic module was turned off, and afterwards vacuum consolidation analysis was performed. Predictive equations for calculating surface settlement due to dynamic compaction and vacuum consolidation was developed using NLR, GEP and MLP. The final models are compared in terms of simplicity of the models and accuracy of the results and their verifications using several validation criteria proposed in the literature. Sensitivity analysis of input variables of the NLR, GEP and MLP models were calculated to determine the significance of each of the variables.

5.2 Numerical simulation of HVDM

Throughout this section, a 3-D model is employed to assess Dynamic compaction in combination with vacuum consolidation, Note that the used mesh is consisting of
polyhedral elements. The mesh size is chosen as 0.35m which is smaller than the critical mesh size. Pan and Selby (2002) suggested the use of dynamic time step of 1e-6sec for the rigid body impact modeling. 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 top view of 3-D finite difference mesh including dimensions and boundary conditions and its elevation view are presented in Figure 5.1 and Figure 5.2, respectively.

Figure 5.1 Top view of 3-D finite difference mesh with dimensions and boundary conditions.
Figure 5.2 Fig. 2 Elevation view of 3-D finite difference mesh.
Boundary conditions are as follows. At the bottom, the normal, strike, and dip quiet boundary, also called damper or absorbent boundary, is applied for avoiding reflection of outward waves. Lateral boundary conditions are free field or infinite. After finishing dynamic impact, non-uniform pore water pressure will be generated inside soil mass. In the next step, dynamic module in Flac3D will be turned off and fluid flow analysis started; vacuum pipe modeled as an elastic pile at the center of mesh that attached to the mesh by Mohr-Coulomb interface and negative pressure applied through leaky boundary around pipe to soil mass.

5.3 Soil constitutive law

In this research, modified Cam-Clay is used for assessing HVDM. The modified Cam-clay model is an incremental hardening/softening elastoplastic 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. In the Cam-clay model, it is assumed that any change in mean pressure is accompanied by elastic change in volume according to Eq.(5-1), therefore, tangent bulk modulus of the Cam-clay material calculated from Eq.(5-2)

\[ \Delta p = \frac{\nu p}{\kappa} \Delta \varepsilon_p^e \]  
\[ K = \frac{\nu p}{\kappa} \]  

Where:

p: mean effective pressure
v: specific volume
\( K \): swelling line slope

\( \varepsilon_p^e \): plastic volumetric strain

\( K \): bulk modulus

5.3.1 Yield Function

The yield function corresponding to a particular value \( p_c \) of the consolidation pressure has the form.

\[
f(q, p) = q^2 + M^2 p (p - p_c)
\]  

(5-3)

Where

\( M \): is a material constant, slope of critical state line,

\( p_c \): consolidation pressure

\( q \): deviatoric stress

\( p \): mean effective pressure

5.3.2 Hardening/Softening rule

The size of the yield curve is dependent on the value of the consolidation pressure, \( p_c \). Eq. (5-4) shows cap hardening rule for Modified Cam-Clay

\[
\frac{dp_c}{de^p} = \frac{p_c}{\lambda - \kappa} (\nu_\lambda - \lambda \ln \left( \frac{p_c}{p_{\text{ref}}} \right))
\]  

(5-4)

It has 4 parameters \( \lambda, \kappa, \nu_\lambda \) and \( p_{\text{ref}} \)

5.3.3 Modeling of vacuum consolidation

For modeling vacuum consolidation, negative pressure should be applied inside of soil mass. For this purpose, leaky boundary condition was applied at the surface of perforated pipes, A leaky boundary has the form

147
\( \text{q}_n = \text{h}(\text{p} - \text{p}_e) \quad (5-5) \)

where \( \text{q}_n \) is the component of the specific discharge normal to the boundary in the direction of the exterior normal (m/sec), \( \text{h} \) is the leakage coefficient in \([\text{m}^3/\text{N}.\text{sec}]\), and it is selected according to discharge capacity of pipe drain. It is supposed that discharge capacity of pipe is 100m3/year, and there isn’t any well resistance; therefore, any high value of \( \text{h} \) is acceptable; \( \text{p} \) is the pore pressure at the boundary surface; in this research a vacuum pressure in the range of -30 kPa to -80 kPa was used and vacuum pressure has a trapezoid distribution. \( \text{p}_e \) is the pore pressure in the region to or from which leakage is assumed to occur, soil mass.

5.4 Methodologies

Three different methods used here for finding correlation between surface settlement and parameters of dynamic compaction and vacuum amplitude. In the following sections this techniques will be elaborated by detail.

5.4.1 Non-Linear Regression Analysis

In the conventional modelling process, regression analysis is a most well-known tool for building a mathematical model. Here, non-linear regression (NLR) based least squares analysis, Ryan (1997) is performed as a classical statistical approach. The NLR method is extensively used in regression analysis primarily because of its interesting nature. This method minimizes the sum-of-squared residuals for each pre-defined equation, accounting for any cross-equation restrictions on the parameters of the system. NLR extends linear regression for use in non-linear real world problem with a much larger and more general class of functions. The simple NLR has a basic form as follows:

\[ y = f(x, \beta) + \varepsilon \quad (5-6) \]
where \( f \) is a pre-defined nonlinear function, \( x \) is the variable vector and \( \beta \) is the unknown coefficient vector. \( \varepsilon \) is error with normal distribution as \( \varepsilon \sim N(0, \sigma^2) \). More detail about nonlinear least squares regression can be found at (Bates and Watts 1988).

5.4.2 Gene Expression Programming

A natural development of gene programming (GP) first invented by Ferreira (2001) is gene expression programming (GEP). Introducing minor changes into the most of genetic operators used in GAs makes them useful in GEP. The five important components of GEP are function set, terminal set, fitness function, control parameters, and termination condition [Alavi and Gandomi, 2011]. A fixed length of character strings is utilized in GEP to represent solutions to the problems. Later, these strings were expressed as parse trees of different sizes and shapes. The name of these trees is GEP expression trees (ETs) Alavi and Gandomi, (2011). The extremely simple creation of genetic diversity at the chromosome level is one advantage of the GEP technique. Inasmuch as the nature of GEP is multi-genetic, it provides the evolution of more complex programs composed of several subprograms. A list of symbols with a fixed length is involved in GEP gene such as any element from a function set like \{+, -, \times, /, \log\} and the terminal set like \{a, b, c, 1\}. The function and terminal sets must have the closure property which means that each function must be capable of taking any value of data type that can be returned by a function or assumed by a terminal, (Alavi and Gandomi, 2011). A typical GEP with the given function and terminal sets can be as below.

\[ +, \times, \log, a, -, +, +, b, a, c, 1, b, c, \]  

(5-7)
in which $x_1$, $x_2$ and $x_3$ are variables and 3 is a constant; for easy reading the element separator of ‘.’ is used. The above-mentioned expression is Karva notation or K-expression, Ferreira, (2001, 2006); Gandomi et al., (2011). A diagram which is an ET can be used to show a K-expression. For instance, the above-mentioned GEP by Eq. (5-6) can be illustrated as Figure 5.3.

![Figure 5.3 Example of ETs.](image)

The conversion begins from the first position in the K-expression corresponding to the root of the ET and reads throughout the string one by one. The GEP shown in Figure 5.3 can also be expressed in the following mathematical form,

$$a ((c + 1) - (b \times c)) + \log (b + a). \quad (5-8)$$

Inversely, recording the nodes of an ET from left to right in each layer, or in other words from root layer down to the deepest one to form the string results in a K-expression. To reach a termination condition, the following steps are used in the algorithm of GEP Ferreira, (2001): (I) random generation of the fixed-length chromosome of each individual for the initial population, (II) expressing chromosomes as ET and evaluating fitness of each individual, (III) selecting the best individuals according to their fitness to
reproduce with modification, (IV) repeating the above process for a definite number of
generations or until a solution has been found.

According to the fitness by roulette wheel sampling with elitism, the individuals are
chosen and copied into the next generation of GEP. The survival and cloning of the best
individual to the next generation are guaranteed by applying the above-mentioned
procedure. By conducting single or several genetic operators on particular chromosomes
including crossover, mutation, and rotation, change in the population is introduced. The
rotation operator is utilized to rotate two subparts of element sequence in a genome with
respect to a randomly selected point, which can also significantly reshape the ETs.

5.4.3 Multilayer perceptron neural network

Same as developing most of conventional statistical models, modeling philosophy
is used to predict nonlinearities in ANNs. Predefined mathematical equations are
employed in the conventional statistical models to extract the relationships between the
model inputs and corresponding outputs. In contradist to the most of available statistical
methods, ANNs work on the basis of the data alone to determine the structure and the
unknown parameters of the model. A class of ANN structures is MLPs based on feed
forward architecture. MLPs as universal approximators are able to approximate
essentially any continuous function with arbitrary degree of accuracy, (Cybenko, 1989).
MLPs are usually used to perform supervised learning tasks including iterative training
methods for adjusting the connection weights within the network. In general, they are
trained with back propagation (BP) algorithm, Rumelhart, Hinton, Williams, (1986). A
schematic diagram of BP neural network has been shown in Figure 5.4. An MLP network
includes an input layer and an output layer. The input layer at least contains one hidden
layer of neurons. Both of input and output layers have several processing units in which each unit is fully interconnected with weighted connections to units in the following layer. Note that, each layer consists of a number of nodes, the interconnection weights of which are multiplied by every input, Alavi, Gandomi, Mollahasani, Heshmati, & Rashed, (2010). Finally, the output \( (h_j) \) is calculated by applying an activation function, \( f \) to the sum of the product,

\[
h_j = f\left(\sum_i x_i w_{ij} + b\right)
\]  

(5-9)

here, \( x_i \), \( w_{ij} \), and \( b \) are the activation of \( i^{\text{th}} \) hidden layer node, the weight of the connection joining the \( j^{\text{th}} \) neuron in a layer with the \( i^{\text{th}} \) neuron in the previous layer, and the bias for the neuron, respectively. In the case of nonlinear problems, the sigmoid functions (Hyperbolic tangent sigmoid or log-sigmoid) are adopted as the activation function. The error function, \( E \) is reduced by adjusting the interconnections between layers.

\[
E = \frac{1}{2} \sum_n \sum_k \left(t_k^n - h_k^n\right)^2
\]  

(5-10)

in which \( t_k^n \) and \( h_k^n \) are the calculated and the actual output values, respectively. Also, \( n \) and \( k \) are the number of samples and the number of output nodes, respectively.
A major constraint in application of MLP is the tendency of trapping in local minima; Hamm et al. (2007). One reliable procedure to cope with this problem is training of the network using global search algorithms such as simulated annealing; Ledesma et al. (2007) which has been successfully applied to complex geotechnical engineering problems; Alavi and Gandomi (2011).

5.5 Results & Discussions

In the following sections results and model development will be discussed, in addition, experimental and predicted values compared too.

5.5.1 Model Development

As mentioned previously, several simulations carried out using FLAC 3D to develop the database. mV0, R, Vac. and Sr were considered as the input variables for the proposed models based on the analysis of soil surface settlement in saturated / partially saturated cohesive soil and after an extensive literature review. The parameter selection of prediction algorithms will affect their model generalization capability especially for
GEP and MLP. For GEP algorithm, the best parameters were selected after a trial and error approach and also according to some previously suggested values in the literature; (Baykasoglu et al. (2008); Gandomi et al. (2012). The optimal parameter settings for the GEP algorithm are presented in Table 5.1. For developing the GEP-based models, a computer S/W program namely GeneXproTools; GEPSOFT (2006) was used.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Settings</th>
</tr>
</thead>
<tbody>
<tr>
<td>General</td>
<td></td>
</tr>
<tr>
<td>Chromosomes</td>
<td>30</td>
</tr>
<tr>
<td>Genes</td>
<td>2</td>
</tr>
<tr>
<td>Gene Size</td>
<td>26</td>
</tr>
<tr>
<td>Linking Function</td>
<td>×</td>
</tr>
<tr>
<td>Function set</td>
<td>+, -, ×, ÷, √, ^2, ^3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Numerical Constants</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Constants per Gene</td>
<td>2</td>
</tr>
<tr>
<td>Data Type</td>
<td>Integer</td>
</tr>
<tr>
<td>Lower Bound</td>
<td>-10</td>
</tr>
<tr>
<td>Upper Bound</td>
<td>10</td>
</tr>
</tbody>
</table>

The performance of an MLP model mainly depends on the network architecture. To avoid overfitting, a single hidden layer network with limited number of neurons is evaluated. It should be noted that the upper bound for the number of hidden neurons is obtained using a strategy proposed by Hecht-Nelson (1987) based on Kalmogorov’s theorem and it is equal to 9 for the current problem. Therefore, the number of hidden neurons is gradually added in this range until the testing results do not improve. Activation function, epochs and learning rate also play important roles in model construction, therefore, several MLP models with different settings were tried to obtain the optimal configurations; Eberhart and Dobbins, (1990). The parameters setting of the MLP algorithm is presented in Table 5.2
Table 5.2 Parameters setting of the MLP algorithm

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Settings</th>
</tr>
</thead>
<tbody>
<tr>
<td>Activation function</td>
<td>log-sigmoid, tan-hyperbolic</td>
</tr>
<tr>
<td>Optimization method</td>
<td>Conjugate-Gradient, Levenberg–Marquardt</td>
</tr>
<tr>
<td>Number of hidden nodes</td>
<td>3-9</td>
</tr>
<tr>
<td>Epochs</td>
<td>500, 1000</td>
</tr>
</tbody>
</table>

Unlike the regression, GP and ANN methods do not have a unique solution and the best model have to be found among several runs with the best parameter setting. The following objective function (OBJ) was proposed previously; Gandomi et al. (2010) as a measure of how well the model predicted output agrees with the experimentally measured output. The selections of the best GEP and MLP models were deduced by the minimization of the following function:

\[
OBJ = \left( \frac{No_{\text{Training}} - No_{\text{Testing}}}{No_{\text{All}}} \right) \frac{MAE_{\text{Training}}}{R^2_{\text{Training}}} + \frac{2No_{\text{Testing}}}{No_{\text{All}}} \frac{MAE_{\text{Testing}}}{R^2_{\text{Testing}}} 
\]

(5-11)

where No.All, No. Training and No.Testing are respectively the number of whole datasets, training and testing data; R and MAE are respectively correlation coefficient and mean absolute error. As is well known, only R is not a good indicator of prediction accuracy of a model. The constructed objective function takes into account the changes of correlation and error together. Higher correlation values and lower error values result in lowering OBJ and, consequently, indicate a more accurate model. Additionally, the above objective function considers the effects of different data divisions for the training and testing data.
For the data division, the developed database was randomly divided into training and testing subsets and several combinations of the training and testing sets were considered to reach a consistent data division. Out of the all samples constructed and simulated herein, 100 of data were used as the training data and the remaining 20 data sets were taken for the testing of the generalization capability of the correlations on the data that played no role in building the models.

5.5.2 NLR-based formulation for HVDM

For the formulation of the soil surface settlement, SS, in terms of mV0, Rt, Vac. and Sr, the following non-linear structure is chosen after a trial and error study:

\[
S_s = \beta(1) \times mV_0^{\beta(2)} \times R_t^{\beta(3)} \times Vac^{\beta(4)} \times Sr^{\beta(5)}
\]  

The obtained coefficients using least squares analysis is presented in Table 5.3. The likelihood of the coefficient is not zero (t-Statistic) and probability of obtaining the estimated coefficient when the actual coefficient is zero (Prob. (t)) ; NLREG (2013) are also presented in Table 5.3. As it can be seen from this table, the likelihood all Prob. (t) is equal to zero which show the coefficients are very significant.

<table>
<thead>
<tr>
<th>Coefficient</th>
<th>Value</th>
<th>t-Statistic</th>
<th>Prob. (t)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\beta(1))</td>
<td>0.007087</td>
<td>6.4045</td>
<td>0</td>
</tr>
<tr>
<td>(\beta(2))</td>
<td>-0.09307</td>
<td>-5.8350</td>
<td>0</td>
</tr>
<tr>
<td>(\beta(3))</td>
<td>0.536313</td>
<td>15.9388</td>
<td>0</td>
</tr>
<tr>
<td>(\beta(4))</td>
<td>0.090951</td>
<td>11.1284</td>
<td>0</td>
</tr>
<tr>
<td>(\beta(5))</td>
<td>0.391526</td>
<td>54.8704</td>
<td>0</td>
</tr>
</tbody>
</table>

With little loss of accuracy, the NLR equation can be simplified and written as:

\[
S_{s,NLR} = 7.5 \times 10^{-3} \left( \frac{mV_0}{R_t} \right)^{0.1} \times Vac^{0.4} \times Sr^{0.5}
\]

A comparison of the actual and predicted flow number for the whole data sets is shown in Figure 5.5
5.5.3 GEP-based formulation for HVDM

The two corresponding expression trees for the final GEP-model are shown in Figure 5.6. The related GEP-based formulation of the soil surface settlement, SS, is as given below:

\[
S_{S,GEP} = \frac{\sqrt[3]{\text{Vac}(\text{Sr} - 6)^{\frac{3}{2}}(\text{Vac} + \text{mV}_0 \times \text{Sr})^{\frac{1}{2}}}}{\text{Sr}/\text{R} + \frac{\text{mV}_0}{100}} \tag{5-14}
\]
Figure 5.6 Expression tree for the GEP model (ETs = Sub-ET$_1$×Sub-ET$_2$)

A comparison the experimental and predicted flow number values for the training and testing data is shown in Figure 5.7.
5.5.4 MLP-based formulation for HVDM

The best model architecture for the formulation of the Ss in terms of mV0, Rt, Vac and Sr is found with 5 hidden neurons. In this study, the explicit MLP-based formulation of soil surface settlement is derived as follows:

\[
S_{S,MLP} = \text{Logsig} \left( \text{Bias} + \sum_{h=1}^{n_h} V_h \text{Logsig} \left( \text{bias}_h + \sum_{i=1}^{n_i} w_{ih} x_i \right) \right)
\]  

(5-15)

where ni and nh are number of inputs and number hidden neurons, respectively. In this formula SS,MLP, x1 to x4 are respectively normalized SS, Rt, Sr, mV0, and Vac values between 0.1 and 0.9. The weights and biases between the input and hidden layer and between the hidden and output layer of the final MLP model are respectively presented in Table 5.4 and Table 5.5.

Figure 5.7 Experimental versus predicted soil surface settlement using the GEP model.
Table 5.4 Weight ($W_{ih}$) and bias values between the input and hidden layer

<table>
<thead>
<tr>
<th>Inputs</th>
<th>Number of hidden neurons ($h$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td>$x_1$</td>
<td>-4.5011</td>
</tr>
<tr>
<td>$x_2$</td>
<td>-0.9032</td>
</tr>
<tr>
<td>$x_3$</td>
<td>3.1583</td>
</tr>
<tr>
<td>$x_4$</td>
<td>2.9034</td>
</tr>
<tr>
<td>bias$_{ih}$</td>
<td>-3.4386</td>
</tr>
</tbody>
</table>

Table 5.5 Weight ($V_{ih}$) and bias values between the hidden and output layer

<table>
<thead>
<tr>
<th>Output</th>
<th>Number of hidden neurons ($h$)</th>
<th>Bias</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>$Y$</td>
<td>2.7000</td>
<td>13.2059</td>
</tr>
</tbody>
</table>

A comparison of the actual and predicted flow number for the training and testing data sets is shown in Figure 5.8.

![Figure 5.8 Experimental versus predicted soil surface settlement using the MLP model](image)

5.6 Model Validity

Smith (1986) concluded that a correlation ($R$) value greater than 0.8 indicates a strong correlation between the predicted and measured values. This is one criterion for
judging model performance. In addition to the correlation criterion, the error values (e.g., RMSE) should be at the minimum. According to these criteria, the proposed ANN and GP models with low RMSE and high R values (see Tables 5 and 7) are able to predict the moment capacity with a high degree of accuracy. The performance of the models on the learning, validation, and testing subsets reveals that it has good predictive ability and generalization performance. Moreover, Golbraikh and Tropsha (2002) suggested some new criteria for external verification of a proposed model on the testing data sets: the slope ($k$ or $k'$) of the regression line between the actual data ($h_i$) and the predicted data ($t_i$) should be close to 1. They also mentioned that the performance indexes $|m|$ and $|n|$ should be lower than 0.1. Recently, Roy (2008) introduced an index ($R_m$) for external predictability evaluation of models; their validation criterion is satisfied for $R_m > 0.5$.

The external validation criteria results for the NLR, GEP and MLP models are shown in Table 5.6. The derived models satisfy all required conditions and are valid models for soil surface settlement during HVDM.
Table 5.6 Statistical parameters of the GEP models for the external validation

<table>
<thead>
<tr>
<th>Item</th>
<th>Formula</th>
<th>Condition</th>
<th>NLR</th>
<th>GEP</th>
<th>MLP</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$R$, Eq. (6)</td>
<td>$R &gt; 0.8$</td>
<td>0.9862</td>
<td>0.9913</td>
<td>0.9980</td>
</tr>
<tr>
<td>2</td>
<td>$k = \frac{\sum_{i=1}^{n}(h_i \times t_i)}{h_i^2}$</td>
<td>$0.85 &lt; K &lt; 1.15$</td>
<td>1.0000</td>
<td>1.0012</td>
<td>0.9999</td>
</tr>
<tr>
<td>3</td>
<td>$k' = \frac{\sum_{i=1}^{n}(h_i \times t_i)}{t_i^2}$</td>
<td>$0.85 &lt; K' &lt; 1.15$</td>
<td>0.9996</td>
<td>0.9985</td>
<td>1.0000</td>
</tr>
<tr>
<td>4</td>
<td>$R_m = R^2 \times (1 - \sqrt{R^2 - Ro^2})$</td>
<td>$R_m &gt; 0.5$</td>
<td>0.8118</td>
<td>0.8540</td>
<td>0.9333</td>
</tr>
<tr>
<td>5</td>
<td>$m = \frac{(R^2 - Ro^2)}{R^2}$</td>
<td>$</td>
<td>m</td>
<td>&lt; 0.1$</td>
<td>-0.0281</td>
</tr>
<tr>
<td>6</td>
<td>$n = \frac{(R^2 - Ro^2)}{R^2}$</td>
<td>$</td>
<td>n</td>
<td>&lt; 0.1$</td>
<td>-0.0281</td>
</tr>
</tbody>
</table>

where

\[
Ro^2 = 1 - \frac{\sum_{i=1}^{n}(t_i - h_i^e)^2}{\sum_{i=1}^{n}(t_i - \bar{t})^2} \quad \text{and} \quad h_i^e = k \times t_i
\]

\[
Ro'^2 = 1 - \frac{\sum_{i=1}^{n}(h_i - h_i^e)^2}{\sum_{i=1}^{n}(h_i - \bar{h})^2} \quad \text{and} \quad t_i^e = k' \times h_i
\]

5.7 Comparison of the NLR, GEP and MLP predictive models

As described above, three different formulas were obtained for the assessment of the soil surface settlement by means of NLR, GEP and MLP methods. The statistical parameters of the NLR, GEP and MLP models including relative MAE, relative root mean square errors (RMSE), R, means, standard deviations (Std. Dev.), and covariances are presented in Table 5.7. Comparisons of the settlement predictions obtained by these models have also been visualized in Figure 5.6 to Figure 5.8. No rational model to predict the soil surface settlement during HVDM has been developed yet; therefore, it was not possible to conduct a comparative study between the results of this research with other studies.
Comparing the performance of the proposed NLR, GEP and MLP relationships, it can be seen from Table 5.7 that MLP has produced the highest correlation and lowest errors and covariance on the whole of data. Therefore, it is the best model in term of prediction accuracy. Comparison between NLR and GEP model, it is clear that the GEP performance is better than NLR in terms of all statistical parameters presented here. However, GEP and MLP-formula have better results than NLR’s, all these three models have very high degree of accuracy and were developed very well.

Considering the simplicity of the models, the NLR formula is the simplest one. The equation obtained by means of the MLP method is very complex and is appropriate to be used as a part of a computer program or via spreadsheet programming. For the GEP equation, it is also short and simple but it is not as simple as the NLR-based equation. It should be noted that both NLR and GEP models can easily be used for routine design practice via hand calculations. From the results and models for this problem, we can say the more complex it is, the more accuracy it has.

### 5.8 Sensitivity Analysis

The sensitivity analysis describes the contribution of the input parameters to the output prediction. For the sensitivity analysis, a procedure proposed by Gandomi (2013) is used here which can be used for any prediction model. Using this method, the

<table>
<thead>
<tr>
<th>Model</th>
<th>$S_{\text{EXP}}$ vs.$S_{\text{PRE}}$</th>
<th>$S_{\text{EXP}}/S_{\text{PRE}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>RMSE (%) MAE (%) R²</td>
<td>Mean Std. Dev. Covariance</td>
</tr>
<tr>
<td>NLR</td>
<td>2.1043 1.6076 0.9726</td>
<td>0.9997 0.0225 0.0225</td>
</tr>
<tr>
<td>GEP</td>
<td>1.6753 1.3830 0.9828</td>
<td>1.0010 0.0178 0.0178</td>
</tr>
<tr>
<td>MLP</td>
<td>0.8011 0.6239 0.9960</td>
<td>0.9999 0.0086 0.0086</td>
</tr>
</tbody>
</table>
percentage of sensitivity for each input parameter is determined using the following formulas, Gandomi et al. (2013):

\[
N_i = f_{\text{max}} - f_{\text{min}}
\]

(5-16a)

\[
S_i = \frac{N_i}{\sum_{j=1}^{n} N_j} \times 100
\]

(5-16b)

where \(f_{\text{min}}\) and \(f_{\text{max}}\) are, respectively, the maximum and minimum of the predicted values over the \(i^{th}\) input domain, where other variables are kept at their mean values.

The results of sensitivity analysis for the proposed NLR, GEP and MLP models are presented in Figure 5.9. As it is seen, the soil settlement is mostly influenced by \(mV_0\).

![Figure 5.9 Sensitivity analyses of the input parameters in the NLR, GEP and MLP models](image)

Figure 5.9 Sensitivity analyses of the input parameters in the NLR, GEP and MLP models
5.9 Conclusion

In this chapter, dynamic compaction and vacuum consolidation simulated numerically in a cylinder domain by Flac3D, for a normally consolidated clay. In addition, this study presents three well-known formulation techniques, namely regression analysis, genetic programming and artificial neural networks for investigating the influence of initial degree of saturation, operational parameters of D.C. and amplitude of applied vacuum on the soil surface settlement. For each method, one of the most promising variant is chosen and used, and their final models are compared in terms of simplicity of the models and accuracy of the results.
CHAPTER VI

DESIGN METHODOLOGY FOR HVDM

6.1 Introduction

The goal of this chapter is developing a numerical model of HVDM in saturated/partially saturated cohesive soil using FLAC 3D in order to achieve a better understanding of the behavior of cohesive soils under dynamic compaction and vacuum consolidation. Generation of non-uniform excess pore water pressure during several drops of tamper impact was modeled. In addition for one model, dissipation of EPWP through vacuum pipes was simulated as well. In addition, an extensive parametric study was performed for investigating the influence of soil properties and operational parameters of D.C. on the responses of soil. Finally a step by step design methodology for HVDM in cohesive soils was developed. It should be mentioned that for considering cycles of vacuum consolidation, water bulk modulus was reduced to 1e6 Pa. Furthermore, it is supposed that vacuum consolidation only reduces initial degree of saturation and increases of mean effective stress, mainly due to vacuum consolidation application is negligible in comparison with deep dynamic compaction.

6.2 Numerical simulation of HVDM

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 research, free fall kinematic energy of tamper transferred to soil surface through Mohr-Coloumb interface. Soil properties, such as strength and stiffness were updated during impact; in every 5 time steps. For modeling large deformation beneath tamper, mesh coordinates were updated in every 5 time steps.

6.2.1 Soil constitutive law

In this research, modified Cam-Clay is used for assessing HVDM. The modified Cam-clay model is an incremental hardening/softening elastoplastic 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; In the Cam-clay model, it is assumed that any change in mean pressure is accompanied by elastic change in volume according to Eq.(6-1), therefore, tangent bulk modulus of the Cam-clay material calculated from Eq.(6-2)

\[
\Delta p = \frac{v_p}{\kappa} \Delta \varepsilon_p^e
\]  \hspace{1cm} (6-1)

\[
K = \frac{v_p}{\kappa}
\]  \hspace{1cm} (6-2)

Where:

- \(p\): mean effective pressure
- \(v\): specific volume
- \(\kappa\): swelling line slope
- \(\varepsilon_p^e\): plastic volumetric strain
- \(K\): bulk modulus
6.2.1.1 Yield Function

The yield function corresponding to particular value \( p_c \) of the consolidation pressure has the form

\[
 f(q, p) = q^2 + M^2 p (p - p_c)
\]

(6-3)

Where

M: is a material constant, slope of critical state line,

\( p_c \): consolidation pressure

q: deviatoric stress

p: mean effective pressure

6.2.1.2 Hardening/Softening rule

The size of the yield curve is dependent on the value of the consolidation pressure, \( p_c \). Eq.(6-4) shows cap hardening rule for Modified Cam-Clay

\[
 \frac{dp_c}{dp} = \frac{p_c}{\lambda - \kappa} \alpha (\nu_{\lambda} - \lambda \ln \left( \frac{p_c}{p_{ref}} \right))
\]

(6-4)

It has 4 parameters \( \lambda, \kappa, \nu_{\lambda} \) and \( p_{ref} \)

6.2.1.3 Cam-Clay Base-Line Parameters

Baseline of Cam-clay parameters used in this research was summarize in Table 6.1 and it is representative of a normally consolidated clay with undrained shear strength of 7kPa at depth of 2m

Table 6.1 Modified Cam-Clay parameters

<table>
<thead>
<tr>
<th>M</th>
<th>( \lambda )</th>
<th>( \kappa )</th>
<th>Reference Pressure(kPa)</th>
<th>Reference Specific Volume(( \nu_{\lambda} ))</th>
<th>Density(( \rho )) Kg/m3</th>
<th>( K_0 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>0.1</td>
<td>0.04</td>
<td>1</td>
<td>2.5</td>
<td>1500</td>
<td>0.6</td>
</tr>
</tbody>
</table>
Dynamic compaction simulation considerations

For modeling dynamic compaction, a 3-D model is employed to assess response of soil mass during impact. Note that the used mesh is consisting of polyhedral elements. Mesh size or element size should be carefully selected for the problem of wave propagation in a semi-infinite half space. Very small element size can cause numerical instability according to a study by Zerwer et al. (2002). On the other hand, large elements cannot allow the shorter waves to travel, which are associated with highest frequencies. The time increment is selected, so that the primary waves, which are the fastest, can be recorded on two consequent nodes along the travel distance. In this research using Flac3D, dynamic time step should be less than critical dynamic time step calculated by Eq. (6-5) and mesh size should be less than Eq. (6-6),

\[ \Delta t_{\text{crit}} = \min \left\{ \frac{V}{C_p A_{\text{max}}^f} \right\} \]  \hspace{1cm} (6-5)

\[ \Delta l \leq \frac{\lambda}{10} \]  \hspace{1cm} (6-6)

Where, \( C_p \), \( V \), and \( A_{\text{max}}^f \) are the p-wave speed, tetrahedral subzone volume, and the maximum face area associated with the tetrahedral subzone, respectively. \( \lambda \) is the wavelength associated with the highest frequency component that contains appreciable energy.

Pan and Selby (2002) suggested the dynamic time step of 1e-6sec for the rigid body impact modeling. 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 0.35m 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 Figure 6.2 and Figure 6.3, respectively. DC impacts could be modeled via applying a force-time history or modeling real impact, by considering tamper as a shell structural element and defining shear and normal parameters of interface for modeling impact between tamper and soil surface by initializing free fall velocity to tamper grid points (nodes) as initial condition of simulation, In this research Mohr-Coulomb interface was employed for modeling real impact, interface parameters are summarized at Table 6.2

A good rule-of-thumb [Flac3D help] is that kn and ks 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:

\[
\max \left[ \frac{(K + \frac{4}{3}G)}{\Delta z_{\text{min}}} \right] = \max \left[ \frac{(5(MPa) + \frac{4}{3}2(MPa))}{0.25m} \right] = 5 \times 10^7 \ (\frac{N}{m^3}) \quad (6-7)
\]

where: K & G are the bulk and shear moduli, respectively; and \( \Delta z_{\text{min}} \) is the smallest width of an adjoining zone in the normal direction. In this research \( k_n \) is taken as 8e8 (N/m^3) and \( k_s \) is taken as 2e8 (N/m^3)

<table>
<thead>
<tr>
<th>shear coupling cohesion (kPa)</th>
<th>shear coupling friction angle (deg.)</th>
<th>Shear coupling stiffness (N/m3)</th>
<th>normal coupling cohesion (kPa)</th>
<th>normal coupling friction angle (deg.)</th>
<th>normal coupling stiffness (N/m3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>10</td>
<td>2e8</td>
<td>7</td>
<td>10</td>
<td>8e8</td>
</tr>
</tbody>
</table>

6.4 Simulation of a real HVDM

Figure 6.1 shows general pattern of HVDM. It usually is conducted in square pattern for both vacuum pipes and dynamic compaction drop points. Simulating effect of all dynamic compaction drops on one vacuum pipe is impossible and unnecessary; in
return, it is precise, considering that every vacuum pipe affected only by 4 impact point around it. Therefore, for simulating a real HVDM, the mesh shown in Figure 6.2 and Figure 6.3 were generated, and dynamic compaction was performed in 4 drop points successively. Dynamic compaction treatment at this simulation comprise 3 impacts per location, impact grid points spaced at 2.0m in a square pattern, tamper radius was 1.0m and its weight was 8 tons. Drop height selected was 10m, hydraulic conductivity value of $10^{-9}$ (m/sec) was selected for this simulation, based on data for normally consolidated clay in literature. Coefficient of volume compressibility, $m_v$, was estimated 0.6 (MPa$^{-1}$).

According to available literature data for normally consolidated soft clay for a representative element at depth 3m, corresponding to a mean effective stress of 50 kPa; according to (Eq. 6-8) $c_h$, $k_h$ and $m_v$ correlated to each other, therefore $c_h$ is equal to 5.25 (m$^2$/year). After finishing 4 successive impact, non-uniform pore water pressure will be generated inside soil mass as shown in Figure 6.4. In the next step, dynamic module in Flac3D will be turned off and fluid flow analysis started. Vacuum pipe was modeled as an elastic pile at the center of mesh, that attached to the mesh by Mohr-Coloumb interface and negative pressure applied through leaky boundary around pipe to soil mass. Boundary conditions was shown in Figure 6.2. For avoiding reflection of outward waves free field boundary used in this simulation.

$$c_h = \frac{k_h}{\gamma_w * m_v} \quad (6-8)$$

Where

$c_h$: horizontal coefficient of consolidation

$k_h$: horizontal hydraulic conductivity

$\gamma_w$: water density
Figure 6.1 General pattern of HVDM
Figure 6.2 top view of generated mesh
Figure 6.3 3D view of generated mesh
Variations of excess pore water pressure vs. depth inside dashed square, Figure 6.4 due to dynamic compaction at 4 impact location was simulated. Change of excess pore water pressure through a vertical section from drop point 1 to drop point 3 was recorded. In addition, Figure 6.4 illustrates contour of generated EPWP inside soil mass due to repeated impacts. These results show how EPWP generated with accumulation of impacts at one drop point and then with impacts at neighbor grid points. As it can be seen maximum EPWP is approximately 30kPa, beneath drop point 1.
Considering generated EPWP as a measure for assessing depth of influence the simulation results for \( W=10 \) ton and \( H=15\text{m} \) shows that DI don’t develop considerably with number of drops and for drop 1 and drop 3 is approximately at the same depth and
there isn’t significant EPWP below depth 6.5m this is in agreement with equation

\[ DI = n \times \sqrt{WH} \]

supposing n=0.5 for soft clay. In addition depth of maximum EPWP typically range from 1/3DI to 1/2DI

Figure 6.5 shows variation of effective stress vs. depth due to D.C. and vacuum consolidation, 0.5 m away from pipe C.L., for tamper weight of 10 ton and drop heights of 10m. As it can be seen, close to 90 kPa for drop height 10m of effective stress increase was observed close to surface after that it decrease, and reaches to approximately 40 kPa at depth of 7m for both drop heights.

![Figure 6.5 variations of effective stress vs depth for W=10ton H=10m and vacuum of -50 kPa](image)

**Figure 6.5 variations of effective stress vs depth for W=10ton H=10m and vacuum of -50 kPa**

6.5 Numerical simulation of HVDM for parametric study

Throughout this section, a 3-D model is employed to assess several impacts during dynamic compaction, as the goal of the first cycle of vacuum consolidation is reducing degree of saturation to near 70%. For considering this cycle in computer simulation, water bulk modulus is reduced to 1e6Pa and coupled hydro-mechanical
simulation of successive impacts performed after that. As the most portion of soil improvement in HVDM occurred during dynamic compaction efforts, and a small part happened during vacuum consolidation; therefore in computer simulation it is supposed that vacuum consolidation acts for reducing degree of saturation and don’t affect strength gain. In addition simulation of transient unsaturated flow inside soil mass is very difficult and time consuming and isn’t necessary for assessing strength gain during HVDM. Note that the used mesh is consisting of polyhedral elements. The mesh size is chosen as 0.35m which is smaller than the critical mesh size. Pan and Selby (2002) suggested the use of dynamic time step of 1e-6sec for the rigid body impact modeling. 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 top view of 3-D finite difference mesh including dimensions and boundary conditions and its elevation view are presented in Figure 6.6 and Figure 6.7, respectively.
Figure 6.6 Top view of 3-D finite difference mesh with dimensions and boundary conditions.
Mesh Side View

5.0 m

Free - Field Boundary

25.0 m

Free - Field Boundary

Normal and Shear Damper

Figure 6.7 Elevation view of 3-D finite difference mesh.
Boundary conditions are as follows: at the bottom, the normal, strike, and dip quiet boundary, also called damper or absorbent boundary, is applied for avoiding reflection of outward waves; lateral boundary conditions are free field or infinite.

6.6 Methodology for calculating undrained shear strength, $S_u$ during HVDM process

In High Vacuum Densification Method, first cycle of vacuum consolidation is performed for reducing initial degree of saturation, for considering this cycle, water bulk modulus set to 1e6 Pa, inside of soil domain; and after that first cycle of dynamic compaction is simulated. For parametric study goal, it is supposed that first cycle of dynamic compaction, comprise of 4 drops, After that, second cycle of vacuum consolidation will start. For considering second cycle of vacuum application, generated excess pore water pressure is initialized to zero and water bulk modulus is set to constant value of 1e6Pa too, After that, second cycle of dynamic compaction will be started. In this part, several drops will be simulated and generation of excess pore water pressure, change of mean effective stress and over consolidation ratio will be recorded. Independent of stress path, history of loading, unloading and reloading and type of loading, drained or undrained conditions, undrained shear strength, $S_u$ is uniquely related to mean effective stress, OCR and Modified Cam-Clay parameters, by the Equation (6-9)

$$c_u = \frac{M}{2} \exp\left[\frac{(r-v_\lambda)}{\lambda} + \ln p'_0 - \left(\frac{\kappa}{\lambda}\right) \ln(OCR)\right]$$

Where:

$$\Gamma = N - (\lambda - \kappa) \ln 2$$

$$v_\lambda = N - (\lambda - \kappa) \ln \frac{M^2 + \rho^2}{M^2}$$

$$OCR = \frac{p_0'}{p_i'} \quad \text{Over-consolidation ratio}$$
\[ p_i' = \left[ \frac{1 + 2k_0}{3} \right] \ast \rho g z \quad (6-13) \]

\[ \eta = \frac{3(1-k_0)}{1+2k_0} \quad (6-14) \]

And the initial specific volume of the sample is calculated from equation (6-15)

\[ v_i = v_\lambda - \lambda \ln \frac{p_0'}{p_{ref}} + \kappa \ln n_p \quad (6-15) \]

Where:

\( M \): slope of critical state line

\( \Gamma \): reference specific volume for critical state line (CSL)

\( v_\lambda \): reference specific volume for normal consolidation line, ncl,

\( \lambda \): slope of normal consolidation line

\( p_0' \): pre-consolidation pressure

\( \kappa \): slope of swelling line

\( n_p \): over-consolidation ratio

\( v_i \): initial in-situ specific volume

\( p_{ref}' \): reference pressure for normal consolidation line

This equation is used for calculating change of undrained shear strength during progress of dynamic compaction.

6.7 Parametric Study

An extensive parametric study using Flac3D code is carried out to examine the effects of influencing factors, such as, dynamic compaction operational parameters and soil properties, during progress of dynamic compaction, on the change of mean effective stress, and OCR, over-consolidation ratio, in normally consolidated cohesive soils. The
goal of this study is correlating change of undrained shear strength, $Su$, to the tamper weight, tamper radius, drop height, number of drops and soil properties.

Metaheuristic techniques have been introduced to solve highly nonlinear and complex modeling and optimization problems. Artificial neural network have (ANN) been known as the most widely used metaheuristic modeling tools for engineering problems. A main constraint of ANN is that it is usually trapped in local minima. Therefore to reach an optimal solution, an ANN should be initially trained using a metaheuristic global optimization algorithm. Ledesma et al. (2009) have recently hybridize ANN and a well-known global optimization algorithm named simulated annealing (SA) to cope this issue and improve the ANN efficiency.

Here, the hybrid ANN and SA, called SA -ANN is utilized to derive the relationship of undrained shear strength, $Su$. This method has been successfully proposed by Alavi and Gandomi (2010) for complex geotechnical engineering problems. The employed hybrid system uses the SA strategy to assign good starting values to the weights of the network before performing optimization. The SA-ANN algorithm is implemented by using the Neural Lab Software for ANNs.

To empirically drive the formula, a database is created containing 1500 datasets. The range of Cam-Clay properties and dynamic compaction parameters in this parametric study is summarized in Table 6.3.
Table 6.3 Parametric study parameters

<table>
<thead>
<tr>
<th>$e_0$ @depth 2.5m</th>
<th>$\lambda$</th>
<th>$\kappa=(1/5)\times$</th>
<th>$M$</th>
<th>$\nu$</th>
<th>$\rho$(kg/m$^3$)</th>
<th>W(ton) Tamper Weight</th>
<th>H(m) Drop Height</th>
<th>Tamper Radius (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.75</td>
<td>0.05</td>
<td>0.01</td>
<td>1.0</td>
<td>0.25</td>
<td>1500</td>
<td>7</td>
<td>10</td>
<td>0.75</td>
</tr>
<tr>
<td>1</td>
<td>0.1</td>
<td>0.02</td>
<td></td>
<td></td>
<td></td>
<td>10</td>
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<tr>
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<td>0.15</td>
<td>0.03</td>
<td></td>
<td></td>
<td></td>
<td>15</td>
<td>20</td>
<td>1.5</td>
</tr>
<tr>
<td>0.2</td>
<td>0.04</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>20</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.3</td>
<td>0.06</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Out of whole datasets 85% was used for training and the rest of them for testing the general capability of the model. The final formula for prediction of mean effective stress and OCR are as follow:

\[ p' = \exp \left( \frac{f_1}{\lambda+0.03} \right) \times p_0' \quad (6-16) \]

\[ f_1 = c_1 \times \exp(-c_2z) \times \frac{\log(WH)}{A} \times \log(N) \quad (6-17) \]

\[ c_1 = 0.76 \times e_0^{1.5} \times \lambda^{0.85} \quad (6-18) \]

\[ c_2 = 1.55 \times \frac{\lambda^{0.55}}{e_0^{0.18}} \quad (6-19) \]

\[ OCR = k_1 \times \exp(-k_2z) \times \frac{\log(WH)}{A} \times \log(N) \quad (6-20) \]

\[ k_1 = 0.47 \times e_0^{1.73} \times \lambda^{0.35} \quad (6-21) \]

\[ k_2 = 0.63 \times \frac{\lambda^{0.77}}{e_0^{0.44}} \quad (6-22) \]

Where:

$p'$: Mean effective stress

$p'_0$: Initial Mean effective stress
z: Depth from soil surface

\( \lambda \): Elastic bulk modulus in cam-clay model

\( p'_0 \): Initial in-situ mean effective stress

c_1: Regression coefficient

e_0: initial void ration

c_2: Regression coefficient

OCR: over-consolidation ratio

k_1: Regression coefficient

k_2: Regression coefficient

6.8 Parametric study Simulation Results

In Figure 6.8 and Figure 6.9, the EPWP within the analyzed geometry calculated via the Modified Cam-Clay constitutive model is given for baseline properties of Cam-Clay, a tamper weight of 10 ton, a drop height of 10 m and tamper radius of 1 m. These figures illustrate contour of the generated EPWP inside soil mass, for first and third drops respectively. As it can be seen, with increasing drop number, influence zone of EPWP extended laterally and vertically it is approximately constant 6.0 m. for drop one and drop three, the maximum generated EPWP is approximately 20 kPa for drop 1 and 31 kPa for drop 3
Figure 6.8 contours of generated EPWP (kPa) for drop 1, and initial degree of saturation of 85%
Figure 6.10 shows contours of total induced vertical stress. It is extended vertically until depth 7m and horizontally to 4m far from C.L. The maximum total vertical stress is 55 kPa beneath tamper,
Figure 6.10 contours of total vertical stress
Figure 6.11 shows contours of horizontal displacement. It can be seen that maximum horizontal displacement occurred at the edges of tamper close to the surface, and the maximum value is 0.12m.

Figure 6.12 shows the contours of vertical displacement. It can be seen that close to the surface and after the edges of tamper, upheaving is happened and the maximum of upheaving is 0.04m.
6.9 Design criteria

A step by step design procedure is proposed here for covering all parameters needed for HVDM design:

1-soil properties should be determined through in-situ and laboratory tests;
2-first cycle of vacuum consolidation performed in the order of one week for reducing
degree of saturation to 70% or less. Spacing between vacuum selected according to permeability, and rate of horizontal consolidation, \( c_h \). Eq. 6-23 can be used for estimating spacing between vacuum pipes, and usually spacing is less than 1m.

\[
U_r = 1 - \exp\left(-\frac{8c_h t}{\mu D^2}\right)
\]  (6-23)

Where:

\( U_r \): degree of consolidation,
\( c_h \): rate of horizontal consolidation
\( t \): time,
\( D \): drainage length, and
\( \mu \): smear parameter, smear effect neglected in this research \( \mu=1.0 \), because vacuum pipes retrieved after finishing project, and disturbance around pipe isn’t considerable.

3- first cycle of dynamic compaction performed according to the required increase in untrained shear strength. Eq. 6-16 through 6-22 will be used for selecting, tamper weight, drop height, tamper Rad. and number of drops. It is supposed that maximum number of drops for first cycle of dynamic compaction is 4.

4- if required strength is not reached another cycle of vacuum consolidation and dynamic compaction should be performed and change of un-drained shear strength should be checked again for required strength match, Eq. 6-16 through 6-22 will be used again for correlating undrained shear strength to dynamic compaction operational parameters.

Here I summarized assumption for preparing this design procedure:

For considering the first cycle of vacuum consolidation, water bulk modulus reduced to 1e6 Pa, and increase of mean effective stress because of vacuum isn’t considered here. And it is conservative, it is supposed that vacuum pressure only decrease initial degree of saturation.
1- For considering second cycle of vacuum consolidation, generated excess pore water pressure during first cycle of dynamic compaction initialized to zero and water bulk modulus remain constant as 1e6 Pa too.

2- Change of mean effective stress and undrained shear strength assessed at C.L. of tamper.

3- It is supposed that first cycle of dynamic compaction have 4 impact.

4- Eq.6-24 used for converting change of mean effective stress to change of undrained shear strength, \( S_u \).

\[
c_u = \frac{M}{2} \exp \left[ \frac{(r - V_0)}{\lambda} \right] + \ln p'_0 - \left( \frac{\kappa}{\lambda} \right) \ln n_p
\]

(6-24)

6.10 Conclusion

HVDM is a new method of soft soil improvement, however there isn’t well-established design method and coupled hydro-mechanical numerical simulations for combination of dynamic compaction and vacuum consolidation in literature, and so further research in this field is needed, in this chapter one real HVDM process simulated by FLAC3D and soil responses like, generated excess pore pressure and change of mean effective stress monitored, in addition one parametric study performed, for correlating change of un-drained shear strength to dynamic compaction operational parameters and soil properties, and a step by step design procedure presented for designing HVDM.
CHAPTER VII

SUMMARY AND CONCLUSION

7.1 Summary

For the objective of developing a design methodology for HVDM in saturated/partially saturated cohesive soils, techniques for numerical simulation of dynamic compaction in combination with vacuum consolidation developed by Cap-Yield and Modified Cam-Clay constitutive models, in addition extensive parametric study using regression analysis, Genetic programming (GP) and Artificial neural network (ANN), performed in this dissertation. Literature review performed for covering previous works done in field of dynamic compaction and vacuum consolidation. Literature review shows that there is very rare numerical simulations and design methodologies for dynamic compaction, in saturated/partially saturated cohesive soils. Coupled hydro-mechanical simulations for dynamic compaction and vacuum consolidation performed in this research and a new methodology for “High Vacuum Consolidation Method” developed in this dissertation work.

In this research FLAC3D a three-dimensional explicit finite-difference program used for simulation purpose. The 3-D model was developed to represent soil continuum in 3-D infinite half space. Initial in-situ horizontal and vertical stresses initialized inside soil mass consistent with dry density, porosity and initial degree of saturation prior to
staring dynamic simulation of dynamic compaction. Tamper impact was modeled using, Modeling real impact, by considering tamper as a shell structural element and defining shear and normal parameters of interface and Applying a normal velocity time-history, directly to the nodes located beneath tamper zone. The FD modeling results in this dissertation was compared with other researchers works in terms of generation of EPWP.

The effect of operational parameters of dynamic compaction, soil properties and initial degree of saturation studied through an extensive FD parametric study. Based on the results from the extensive FD parametric study, several correlation equations suggested for estimating generation of EPWP in 1-D and 3-D domain, surface settlement due to vacuum consolidation in combination with dynamic compaction, and change of mean effective stress and un-drained shear strength during vacuum consolidation process.

7.2 Conclusion

Based on the research done here, the following conclusions can be drawn.

- Cap-Yield constitutive model is very good one, for modeling coupled hydro-mechanical dynamic compaction and simulating high level of plastic deformation and stress.
- Flac 3D code could be successfully used for modeling geotechnical phenomena, like dynamic compaction
- The parametric study conducted to find predictive equation for EPWP generation in 1-D and 3-D domain, surface settlement during HVDM and change of mean effective stress and un-drained shear strength during HVDM process.
- The model compare with other researchers work for generation of EPWP
• Design methodology developed for HVDM

• For high impact energy and high initial degree of saturation, say more than 95%, negative EPWP generated beneath tamper.

• For obtaining better results, during dynamic compaction simulation; in addition to soil stiffness and strength, water bulk modulus should be updated in every few step.

• Finally a step by step design methodology developed for designing “High Vacuum Consolidation Method” selecting tamper operational parameters during HVDM process

7.3 Recommendation for future works

• More comprehensive modeling of transient water flow during vacuum consolidation process in saturated/partially saturated cohesive soils should be a focus of future works.

• Additional investigation should be performed on coupled hydro-mechanical simulation of adjacent drops or overlapping drops for better understanding of interaction mechanism between the adjacent drop points.

• For modeling high level of distortion at the edges of tamper and beneath tamper, for high level energy of impact, to avoid error it is good idea that very distorted elements constitutive model converted to null, by writing a function for that

• With the increasing computational power, dynamic analysis and vacuum consolidation analysis could be run simultaneously.
• More field data and laboratory tests are needed to validate numerical simulation results and design methodology
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