Aalto University
School of Engineering

Michael Sandelin

Evaluation of Dynamic Compaction Method and Rapid Impact Compaction Method for Soil Improvement

Master's Thesis
Aalto University
School of Engineering
Department of Civil Engineering

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Supervisor: Prof. Wojciech Solowski, Aalto University
Advisor: M.Sc. Pertti Heininen, Graniittirakennus Kallio Oy

Aalto University
School of Engineering

Aalto University, P.O. Box 11000, 00076 AALTO
www.aalto.fi

Abstract of Master's Thesis

Author Michael Sandelin	
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Abstract

A soil improvement method with capacity to densify a larger amount of mass within a shorter period of time is highly appreciated as it could help to reduce the working time. Deposits of up to several meters thick soil can be complicated to compact homogeneously and accurately. For a vibratory roller or similar, it would be required that deposits are compacted in several phases by filling and compacting alternately. This may, on the other hand, be very time consuming. Therefore, compaction methods that can offer a performance ability to the desired level within only one phase are highly coveted. Among such methods are, for instance, the conventional falling weight compaction method, also known as dynamic compaction (DC), and the relatively recently developed rapid impact compaction (RIC) method.

The thesis subject is a result of industrial interest in rapid impact compaction method. Industry is keen to try this innovative method, yet there is uncertainty whether the significant cost of the machines would lead to improvements in the work routines and efficiency justifying the cost. Hence, the thesis investigates how the dynamic compaction and rapid impact compaction compares, how the compaction energy in both methods affects the soil improvement results and, finally, how economical the rapid compaction is, when compared to the dynamic compaction.

Prescribed guidelines along with previous performed compaction projects around the world connect the thesis to a complete analysis of the methods. Parallels have been drawn between the proposed directives and case studies, as well as the projects have been interpreted between each other. This has made it possible to demonstrate when the present methods are favourable to apply and, on the other hand, more unfavourable. The reader can also receive a basic knowledge of the associated soil mechanics as well as common geotechnical insights in the behaviour of soil at exposed state. This will give the reader a considerably better perception in the description of the methods.

Keywords dynamic compaction, falling weight compaction, rapid impact compaction, soil improvement, densify

Aalto-universitetet
Högskolan för ingenjörsvetenskaper

Aalto-universitetet, PB 11000, 00076 AALTO
www.aalto.fi

Sammandrag av diplomarbetet

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Sammandrag

En markförbättringsmetod med kapacitet att packa en större mängd massa inom en kortare tidsperiod är högt värderat, eftersom det kan hjälpa att reducera arbetstiden. Markskikt med upp till flera meter tjocka jordmassor kan vara komplicerade att få enhetligt och noggrant packade. För en vibrationsvält eller dylikt skulle det krävas att markskikten packas i flertalet omgångar genom att fylla och packa om vartannat. Detta kan å andra sidan vara väldigt tidskrävande. Därför är komprimeringsmetoder, som kan erbjuda en prestationsförmåga till den begärda nivån inom endast en omgång, mycket eftertraktade. Till sådana metoder hör bland annat den traditionella fallviktskomprimeringsmetoden, även kallad dynamisk komprimering, samt den relativt nyligen utvecklade snabbslagskomprimeringsmetoden.

Avhandlingsämnet är ett resultat av ett arbetsmarknadsintresse i snabbslagskomprimeringsmetoden. Branschen är angelägen om att prova denna innovativa metod. I nuläget finns en ovisshet kring huruvida den betydelsefulla kostnaden av maskinerna skulle leda till förbättringar i arbetsrutinerna och effektiviteten, som rättfärdigar kostnaden. Därmed undersöker examensarbetet hur fallviktskomprimeringen och snabbslagskomprimeringen jämförs, hur komprimeringsenergin i båda metoderna påverkar markförbättringsresultatet och, slutligen, hur ekonomisk snabbslagskomprimeringen är, när den jämförs med fallviktskomprimeringen.

Föreskrivna riktlinjer tillsammans med tidigare utförda komprimeringsprojekt världen över knyter samman examensarbetet till en komplett analysering av metoderna. Paralleller har dragits mellan de anförda direktiven och fallstudierna, tillika som projekten sinsemellan har tolkats. Detta har gjort det möjligt att kunna påvisa när de föreliggande metoderna är gynnsamma att tillämpa och, å andra sidan, mindre ändamålsenliga. Läsaren tillhandahålles även en grundläggande vetskap kring den tillhörande jordmekaniken samt gängse förekommande geotekniska insikter i markens beteende vid utsatt tillstånd. Detta skall ge läsaren en avsevärt bättre uppfattning i samband med beskrivningen av metoderna.

Nyckelord dynamisk komprimering, fallviktskomprimering, snabbslags-komprimering, markförbättring, packa

Aalto-yliopisto
Insinööritieteiden korkeakoulu

Aalto-yliopisto, PL 11000, 00076 AALTO
www.aalto.fi

Diplomityön tiivistelmä

Tekijä Michael Sandelin	
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Tiivistelmä

Maanparannusmenetelmä, joka kykenee tiivistämään suuremman massamäärän lyhyemmässä ajassa, on erittäin arvostettu, koska se voi vähentää vaadittua työaikaa. Maakerrokset, jossa maamassoja on jopa useita metrejä, voivat olla monimutkaisia saada yhtenäisesti ja virheettömästi tiivistettyä. Täryjyrältä tai vastaavalta vaadittaisiin, että maakerrokset tiivistetään useaan otteeseen täyttämällä ja tiivistämällä vuorotellen. Tämä saattaa olla aikaa vievää ja siksi tiivistysmenetelmät, jotka tarjoavat suorituskykyä vaaditulle tasolle, ovat hyvin toivottuja. Tällaisiin menetelmiin kuuluvat mm. perinteinen pudotustiivistysmenetelmä, joka tunnetaan myös nimellä dynaaminen tiivistys, ja suhteellisen äskettäin kehitetty nopeaiskutiivistysmenetelmä.

Opinnäyteaihe on tulos nopeaiskutiivistysmenetelmän työmarkkinakiinnostuksesta. Toiminta-ala on innokas kokeilemaan tätä innovatiivista menetelmää. Tällä hetkellä on epävarmuutta siitä, johtaisiko koneiden merkittävät kustannukset parantamaan työn rutiineja ja tehokkuutta, perustellakseen kustannukset. Täten opinnäytetyö tutkii kuinka nopeaiskutiivistystä ja pudotustiivistystä verrataan, miten kummassakin menetelmässä oleva tiivistysenergia vaikuttaa maaperän parannustuloksiin ja lopuksi, kuinka taloudellinen nopeaiskutiivistys on verrattuna pudotustiivistykseen.

Suositellut ohjeet sekä aikaisemmat suoritetut tiivistysprojektit ympäri maailmaa yhdistävät opinnäytetyön menetelmien täydelliseen analyysiin. Ehdotettujen direktiivien ja tapaustutkimusten välillä on tehty rinnastuksia, ja hankkeita on tulkittu keskenään. Tämä on mahdollistanut sen, että voidaan osoittaa koska nykyiset menetelmät ovat suotuisat käytettäväksi, ja toisaalta vähemmän tarkoituksenmukaiset. Lukijalle annetaan myös mukaan kuuluvan maamekaniikan perustaidot sekä yleisesti esiintyvät geotekniset tiedot maan käyttäytymisestä altistetussa tilassa. Tämä antaa lukijalle huomattavasti paremman käsityksen menetelmien kuvauksessa.

Avainsanat dynaaminen tiivistys, pudotustiivistys, nopeaiskutiivistys, maanparannus, tiivistää

Preface

This master's thesis has been completed summer 2018 in conjunction with the completion of my master's program in Geoengineering at Aalto University, Espoo. The thesis has been done in cooperation with the civil engineering company Graniittirakennus Kallio Oy, Vantaa. I came in contact with Graniittirakeenus Kallio Oy the summer before, when I worked for them. During that time, I was granted a thesis topic. I gave them a number of alternative topics that interested me, which later led to this thesis.

After completion of the thesis, I would like to thank the responsible people at Graniittirakennus Kallio Oy, who made the suggestion for thesis topic and gave me the opportunity to accomplish the project. The financial support they awarded me was of a great pleasure to receive, while it also was a contributing incentive to finish the thesis within a reasonable time. Further, I would like to thank the advisor Pertti Heininen from Graniittirakennus Kallio Oy for the available help he offered during the fulfilment. Finally, I am also very grateful for all the advices and knowledge received from the supervisor Wojciech Solowski at Aalto University.

Espoo, 12 September 2018

Michael Sandelin

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Nomenclature

Latin letters

A	Contact area	m^2
A_e	Influence area of each impact point	m^2
AE	Applied energy at each drop point	ton-m/m^2
a_Z	Depth correlation coefficient	-
b_Z	Depth correlation coefficient	-
c'	Cohesion	MPa
c_u	Undrained cohesion	MPa
d	Distance from impact point	m
D_i	Depth of improvement	m
e	Void ratio	-
e_{max}	Void ratio at loosest state	-
e_{min}	Void ratio at densest state	-
g	Gravitational acceleration (9,81 m/s^2)	-
H_d	Drop height	m
I_D	Density index (or Relative density)	-
I_L	Liquidity index	-
I_P	Plasticity index	-
M	Momentum	ton-m/s
M	Total mass	kg
M_a	Mass of gas (air)	kg
M_s	Mass of solids (grains)	kg
M_w	Mass of liquid (water)	kg
n	Porosity	%

N'	Interparticle force	N
n_c	Empirical coefficient	-
N_d	Number of drops	Pcs.
P	Normal force	kN
P	Number of passes	Pcs.
PPV	Peak particle velocity	m/s
q_{fu}	Deviator stress at failure	MPa
r_o	Radius of tamper	m
s	Drop spacing	m
S_r	Water saturation	-
T	Tangential force	kN
u	Pore water pressure	MPa
v	Impact velocity	m/s
V	Total volume	m^3
V_a	Volume of gas (air)	m^3
V_s	Volume of solids (grains)	m^3
V_v	Volume of voids	m^3
V_w	Volume of liquid (water)	m^3
w	Water content	%
W	Weight	kN
w_L	Liquid limit	%
w_P	Plastic limit	%
w_S	Shrinkage limit	%
W_s	Weight of solids (grains)	kN
W_t	Mass of drop weight (tamper)	ton
W_w	Weight of liquid (water)	kN

z	Depth	m

Greek letters

γ	Unit weight (or Specific weight)	kN/m^3
γ'	Buoyant unit weight	kN/m^3
γ_d	Dry unit weight	kN/m^3
γ_s	Compact unit weight	kN/m^3
γ_{sat}	Saturated unit weight	kN/m^3
γ_w	Unit weight of water	kN/m^3
ρ	Bulk density	ton/m^3
σ	Total normal stress	MPa
σ'	Effective normal stress	MPa
σ_f'	Effective normal stress at failure	MPa
τ	Shear stress	MPa
τ_f	Shear strength	MPa
τ_{fu}	Undrained shear strength	MPa
ϕ'	Angle of shearing resistance	deg

1 Introduction

There is a continual growing competition between today's contractors. The timetable is tightened at the site and it requires specialized soil improvement techniques to the same extent as alternative filling methods become faster in their kind. Compaction methods should be able to densify a thicker layer within a shorter period of time (Cofra: Documents, CDC compaction). Therefore, it is a constant reason for companies to be unique in their niche, come up with innovative methods that make a difference in the field.

The thesis work is about an evaluation of two different impact oriented ground improvement method. One is the conventional falling weight compaction method, perhaps more known as dynamic compaction method (DC). The method is a simple dynamic compaction method that has been available far back in the past allowing a weight to freely fall from a certain height. The other, which is a more recently developed compaction method, is the rapid impact compaction method (RIC). The method is a fast and intensive soil improvement method that performs a more controlled compaction work than DC. The evaluation will further give the opportunity to consider whether any possible commissioning of the RIC method would be beneficial for the industry, or should it still prefer the already long used DC method.

1.1 Background

The more traditional DC method has been used for a quite long time as an alternative soil improvement method in the fields. This has led to the fact that the industry is keen to try the more recently developed RIC method. There is yet an uncertainty whether the significant cost of the machines would lead to improvements in the work routines and efficiency justifying the cost. Therefore, it is a demand to investigate if RIC would be a significantly better technique, when it comes to those issues. Both the working rate and the performed work must be taken into account.

Because RIC has not really established itself in these areas, the main shortcomings are finding useful and appropriate data on previously implemented soil improvements. However, there are manufacturers of these devices outside the national borders that sit on a variety of good and helpful test data about the device. This is something that will be of great use during the course of work. At the same time, there are a large number of useful sources online in the form of previous degree projects, publications and reports made by both knowledgeable individuals and companies. These texts will be of great importance during the progress of the work. In addition, the surroundings are filled with experienced and formed individuals who possess some information on the subject.

1.2 Purpose and Goal

The purpose of the thesis work is to investigate if the industry could benefit from introducing RIC as a possible soil improvement method. From an industrial interest, it is desired to obtain a thorough evaluation of the traditional DC method and the recently developed RIC method. The comparison will be based both on an economic and a performance perspective.

The review hopes to provide a clear picture of how these two soil improvement methods are suitable for different soil conditions, both nationally and generally. As well as costly, how much more efficient one is compared to the other. With a successful job, there will be an opportunity to make a decision whether it is worth investing in any of these soil improvement methods or not.

1.3 Research Questions

The most important questions and problems are as follows:

- How rapid impact compaction compares to the dynamic compaction?
- How compaction energy affects the soil improvement results?
- How economical rapid impact compaction is, when it compares to dynamic compaction?

1.4 Methods

The implementation of the work is basically only based on previously performed work and studies on the subject. In this way, it has been considered finding the largest amount of practical data about these two soil improvement methods. The aim has been to come across as many different studies as possible, which concern soil improvement works within various soil types. However, useful reports from companies in the area have also been included. These are something that is valued very high among the sources. In addition, interviews have been held together with experienced and knowledgeable people in the subject.

On the other hand, a thorough literature study has been carried out for the touching parts of soil improvement. A large number of previous books and writings on the principles of soil mechanics and foundation engineering have been studied. While all criteria are entirely based on references to guidelines and standards.

1.5 Limitations

The scope of the work can more or less be done in a relatively large-scale way. Several different soil improvement methods can be taken into consideration. By doing so, see which methods are the most profitable and so on. The idea was initially that it would be investigated how RIC relates to soil improvement methods such as a normal mass exchange, deep stabilization, etc. It soon came to the conclusion that this would be an excessive work, thus limiting the work to only carry out an evaluation between DC and RIC.

2 Science of Soil Materials and Mechanics

It is essential to know the underlying science of the soil materials and some basic soil mechanics, when performing soil improvement and in this case dynamic compaction. Changes occur in soil structure down to the constituent level. Deformations and stresses arise in each layer under loading (Axelsson & Mattsson, 2016). In this chapter, it will be clarified the touching parts that concern the soil improvement technique. Everything from the constituents of the soil as well as the structure to some soil mechanics with regard to compaction is discussed.

2.1 Fundamental Quantities Describing of Soil

Soils consist principally of three different elements, which are solids, pore water and pore gas. There can also be only two-phase composition. However, this means that the soil is a so-called multiphase system consisting of solid, liquid and gaseous phases. There the solid phase is the actual skeleton of the soil, which is grains (or particles). The grains can consist of mineral soils or organic soils. Between the grains is the pore space or also called voids that mainly include water, but there can also be some air in between. In Figure 2-1 a visualization has been designed of how a natural soil could look like, while Figure 2-2 demonstrates a schematic figure of soil's constituents (Axelsson & Mattsson, 2016; Craig, 2004).

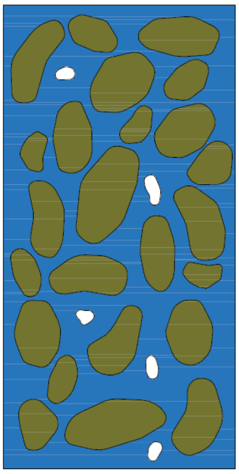


Figure 2-1: Natural soil structure

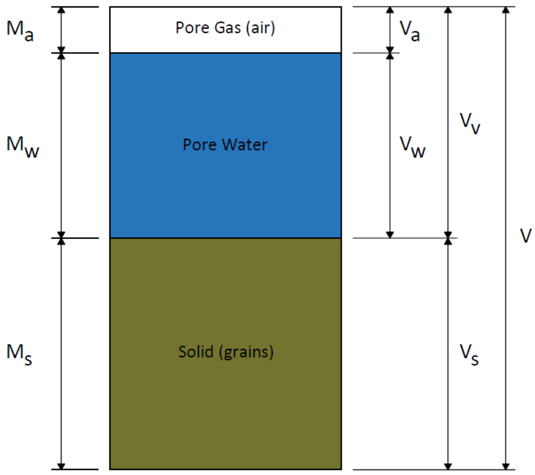


Figure 2-2: Schematic figure of soil's constituents

The composition, shape and size of grains can vary quiet much, as well as the size of the pore space and the water content in the voids. This is something that affects the properties of the soil in different aspects. It is of great importance to find out the properties of the soil in terms of fracture properties and strength, bearing capacity, deformation properties and stability of the skeleton. The following subheadings 2.1.1, 2.1.2 and 2.1.3 show some basic concepts and definitions that play a decisive role in the relationship between the different phases (Axelsson & Mattsson, 2016; Mitchell & Soga, 2005).

2.1.1 Density and Unit Weight

The density tells quite much about what kind of the character of the soil it is about. There the mass and volume will be significant. In this context, the mass and volume have gotten specific indexes for the different phases just to separate them, *s* stands for solid, *w* for water and *a* for air. The *bulk density* is the most common density and usually refers to the density present in the naturally moist state (in-situ), see equation (2.1), (Axelsson & Mattsson, 2016).

$$\rho = \frac{M}{V} \tag{2.1}$$

where:

ρ = bulk density

M = total mass ($M = M_s + M_w$)

V = total volume ($V = V_s + V_v$)

Within the geotechnical terminology, the word density does not appear in everyday speech. It is mainly about *unit weight* (or specific weight), γ. Therefore, instead of the term density the unit weight will be found in these geotechnical applications. The unit weight refers to weight W (a force) per volume unit, which means that the current density is multiplied by gravitational acceleration g and gets the unit kN/m^3 (Axelsson & Mattsson, 2016; Craig, 2004).

The unit weight may also occur in other states depending on the circumstances. It will just get a different designation. There are cases where the soil sample consists of a total dry soil. The weight represents only of the solids (grains) and it is called *dry unit weight* (2.2). Then there are situations that are other way around, where the soil sample is completely water saturated. This means the whole pore space (voids) is completely filled with pore water. The phenomenon is called *saturated unit weight* (2.3) (Axelsson & Mattsson, 2016).

$$\gamma_d = \frac{W_s}{V} \tag{2.2}$$

where:

γ_d = dry unit weight

W_s = weight of solids

V = total volume

$$\gamma_{sat} = \frac{W_s + W_w}{V} \tag{2.3}$$

where:

γ_{sat} = saturated unit weight

W_s = weight of solids

W_w = weight of water

V = total volume

A soil mass, taking place under the ground water surface there the particles are exposed to up thrust, is named as a *buoyant unit weight* (2.4). The principle is supported directly by Archimedes. Finally, there is also a designation for the solids known as *compact unit weight* (2.5). The particles of the Nordic mineral soils have a compact unit weight of 26 - 27 kN/m^3 (Craig, 2004; Axelsson & Mattsson, 2016).

$$\gamma' = \gamma_{sat} - \gamma_w \tag{2.4}$$

where:

γ' = buoyant unit weight

γ_{sat} = saturated unit weight

γ_w = unit weight of water

$$\gamma_s = \frac{W_s}{V_s} \tag{2.5}$$

where:

γ_s = compact unit weight

W_s = weight of solids

V_s = volume of solids

2.1.2 Void Ratio, Porosity and Relative Density

The pore volume can either be categorized as the ratio of the volume of solids (2.6) or as the ratio of the total volume (2.7). This tells about a soil's *void ratio* and *porosity* respectively. There the void ratio is given as a decimal and the porosity as a percentage (Craig, 2004).

$$e = \frac{V_v}{V_s} \tag{2.6}$$

where:

e = void ratio

V_v = volume of voids

V_s = volume of solids

$$n = \frac{V_v}{V} \tag{2.7}$$

where:

n = porosity

V_v = volume of voids

V = total volume

Depending on how well the particles are compacted, an identical soil type may prove to have different pore volumes. In turn, it shows a higher and lower void ratio. Out of that, a relationship between the in-situ void ratio and the limiting void ratios provides the *density index* (or relative density). The relative density relates well to other properties. However, it is mostly applied for cohesionless soil. The equation (2.8) defines the relative density (Axelsson & Mattsson, 2016; Mitchell & Soga, 2005).

$$I_D = \frac{e_{max} - e}{e_{max} - e_{min}} \tag{2.8}$$

where:

I_D = density index

e = void ratio of a soil sample

e_{max} = void ratio in the loosest state

e_{min} = void ratio in the densest state

The relative density may vary from 0 at the loosest state to 1 at the densest state, see Figure 2-3 (Craig, 2004).

a) b)

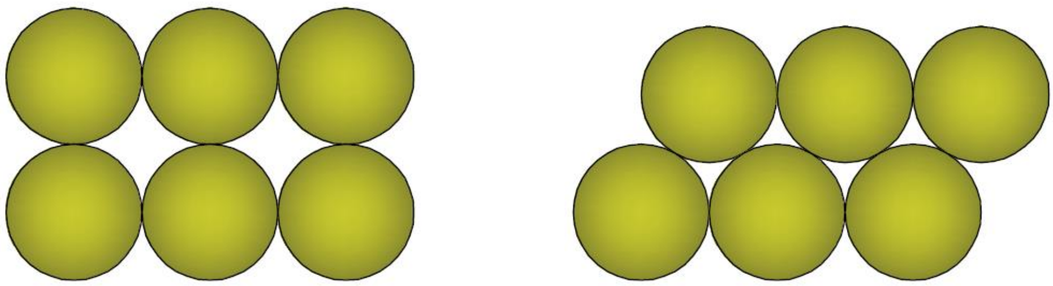


Figure 2-3: a) Loosest and b) densest state of an ideal grainy mass

2.1.3 Water Content and Saturation

The proportion of pore water in voids is something that can both be described as a relationship between masses or as a relationship between volumes. On the other hand, the two have their own definition of the characteristics of a soil. *Water content* is defined by the ratio of the pore water mass to the mass of the solids, (2.9). This tells about the amount of water present in the soil and is very significant for the behaviour of a soil. The *water saturation*, in turn, expresses the relationship between the pore water volume and the volume of the pore space, (2.10). It describes how much of the pore volume is filled with water. A water saturated soil is defined, when the voids are completely filled with water (Craig, 2004).

$$w = \frac{M_w}{M_s} \tag{2.9}$$

where:

w = water content

M_w = mass of water

M_s = mass of solids

$$S_r = \frac{V_w}{V_v} \tag{2.10}$$

where:

S_r = water saturation

V_w = volume of water

V_v = volume of voids

2.2 Soil Classification

There are a lot of different types of soil, which means that the composition and characteristics of the soil vary pretty much. Thus, soil can be divided into several different groups with respect to various classifications such as *organic content*, *consistency*, *frost susceptibility*, *formation* and *strength properties*. However, the most common and most important classifications are *grain size* and *grain size distribution* (Axelsson & Mattsson, 2016).

2.2.1 Grain Size and Grain Size Distribution

A classification of the grain size puts the soil into different fractions, see Table 2-1. This ranges from particles of clay less than 2 µm up to big boulders greater than 200 mm. A soil that occurs in-situ state is usually consisting of several fractions. This creates a grain size distribution and is determined by sieving. The soil is divided fractionally in a sieving machine according to size and amount. The result is plotted on a distribution graph where the horizontal axis indicates the particle size in a logarithmic scale and the vertical axis indicates the proportion of particles smaller than the actual size in percentage, see Figure 4-7 at page 25. The plotted result forms a distribution curve that tells us about the soil type (Axelsson & Mattsson, 2016; Craig, 2004).

Table 2-1: Grain fractions for mineral soils [1]

Fraction group	**Fraction**	**Designation**	**Grain size mm**
Very coarse-grained	Boulder	Bo	> 200
	Cobble	Co	> 63 to 200
Coarse-grained	Gravel	Gr	2 to 63
	Sand	Sa	0.063 to 2
Fine-grained	Silt	Si	0.002 to 0.063
	Clay	Cl	≤ 0.002

2.2.2 Consistency

A fine-grained soil is very dependent on the water content in the view of their engineering properties (Axelsson & Mattsson, 2016). The consistency of these soils may vary in four different states, ranging from liquid, plastic, semi-solid and solid state depending on the water content. At specific water content of soil a transition between the states is going to occur. These transitions, which are *shrinkage limit* (w_S), *plastic limit* (w_P) and *liquid limit* (w_L), differ from case to case depending on the soil. However, most of the fine-grained soils are in the plastic state. Plasticity only exists with a perceptible content of clay mineral particles (Craig, 2004).

The range of the plastic state (or plasticity) is defined as plasticity index. This expresses the difference between liquid limit and plastic limit, as shown in the equation (2.11). In addition, a liquidity index can be determined by the equation (2.12) (Axelsson & Mattsson, 2016).

$$I_P = w_L - w_P \tag{2.11}$$

where:

I_P = plasticity index

w_L = liquid limit

w_P = plasticity limit

$$I_L = \frac{w - w_P}{w_L - w_P} \tag{2.12}$$

where:

I_L = liquidity index

w = water content

w_P = plasticity limit

w_L = liquid limit

2.3 Effective Stress and Shear Stress

Both an unloaded and loaded soil mass represent forces in their contact points between the particles (interparticle contact). For unloaded soil masses, the forces are completely dependent on the self-weight. While for loaded soil masses, the external load contributes also additional forces in the points of interparticle contact. A soil mass, with a contact area A, exposed by a vertical force (normal force P), is going to develop a total normal stress σ. The expression can be seen in equation (2.13) as well as with reference to Figure 2-4. If then imagining the soil mass being completely dry. The interparticle forces N' will solely spread the normal force vertically in the soil mass, as the air filled voids cannot transfer any normal force. In order to indicate the transmission only occurs through the points of interparticle contact, the stress is termed as an effective normal stress σ'. The effective normal stress is presented in the equation (2.14) (Axelsson & Mattsson, 2016; Craig, 2004).

$$\sigma = \frac{P}{A} \tag{2.13}$$

where:

σ = total normal stress

P = normal force

A = contact area

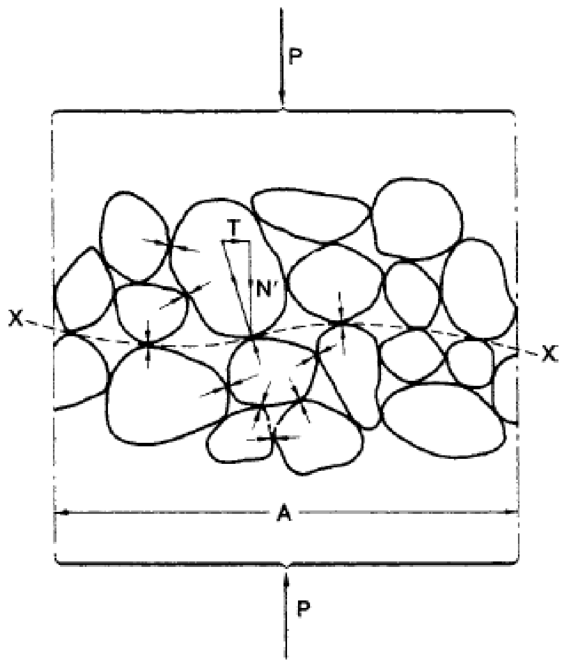


Figure 2-4: A cross-section of a soil mass [1]

$$\sigma' = \frac{\Sigma N'}{A} \tag{2.14}$$

where:

σ' = effective normal stress

N' = interparticle force

A = contact area

On the other hand, if the voids are filled with pore water, a so-called fully saturated soil. An additional pore water pressure is going to act over the entire area transferring the load. Equation for pore water pressure is shown in (2.15). The total normal stress gives thus a new expression according to equation (2.16) (Axelsson & Mattsson, 2016).

$$u = \gamma_w \cdot z \tag{2.15}$$

where:

u = pore water pressure

γ_w = unit weight of water

z = depth

$$\sigma = \sigma' + u \tag{2.16}$$

where:

σ = total normal stress

σ' = effective normal stress

u = pore water pressure

Since it is primarily talked about the effective stress as well as the most natural way of calculating a stress analysis of a soil is to first determine total stress and pore water pressure (Axelsson & Mattsson, 2016). The equation for effective stress is going to sound as follows:

$$\sigma' = \sigma - u \tag{2.17}$$

The contact points are not only exposed to a normal force but also a tangential force T, see Figure 2-4. Thus, shear stresses that appear in the cross-section can be defined by equation (2.18). Since the pore water cannot transfer any shear forces, the effective components remain the same as the total components (Axelsson & Mattsson, 2016).

$$\tau = \frac{\Sigma T}{A} \tag{2.18}$$

where:

τ = shear stress

T = tangential force

A = contact area

2.4 Compression of soil

Compression of a soil (reduction in the volume) means that the density will increase as the particles are forced into a denser state. The volume of the soil skeleton changes, when the particles are rearranged (Craig, 2004). The particle sizes, grain size distribution and shape of the particles in the soil skeleton are related to the engineering properties such as strength, compressibility and permeability (Mitchell & Soga, 2005). The compressibility of a soil can further vary considerably regarding the amount of water. A material containing a large amount of water in the pores is much more difficult to compress compared to a material containing the same amount of air instead. Because the water is a very incompressible material, while the air is much easier to compress. The particles have in such case easier to be rearranged. Thus, a compression of the soil occurs only if the water has somewhere to go (Craig, 2004).

A water saturated soil mass exposed to compression or an external load increases its total stress. This also leads to a drainage wants to take place and a change in the pore pressure occurs. A compression of the soil (volume reduction) during water drainage, is usually called consolidation. Consolidation itself can manifest in several states depending on whether the ground is loaded or unloaded, but is most relevant in fine-grained soils. Figure 2-5 shows a typical plot of strain ε after consolidation against effective stress σ'. The soil under compression behaves like a non-linear elastoplastic body. A loaded soil, which after some time achieves a certain compression strain, is said to be *normally consolidated* when exposed to its historically highest effective stress. This occurs in point 1 in Figure 2-5 a). At a possible reduction in total stress (unloading of the soil), the soil will swell slightly. For a fine-grained soil slightly more than for a coarse-grained soil. The swelling, which is reverse of consolidation, shows that the soil tends to return somewhat to its original shape, refer to point 2 in Figure 2-5 a). This part of the compression strain reveals thus to be reversible, i.e. elastic. However, most of the compression strain is irreversible, i.e. plastic. This shows that the soil is elastoplastically compressed. The unloaded soil, which receives an effective stress less than the historically highest effective stress, is said to be *overconsolidated*. The highest effective stress that an overconsolidated soil has ever obtained is called *pre-consolidation pressure*. If the overconsolidated soil (unloaded soil) is recompressed again. The pre-consolidation pressure will again be achieved, but the compression of soil is considerably stiffer than in the normally consolidated state. As soon as the pre-consolidation pressure is achieved, like in point 3 in Figure 2-5 b), the normal consolidation line NCL (or virgin compression line) will return to its original shape as in the normally consolidated state. A *fully consolidated soil* will occur when the soil body stays in equilibrium, see point 4 in Figure 2-5 b) (Axelsson & Mattsson, 2016; Craig, 2004).

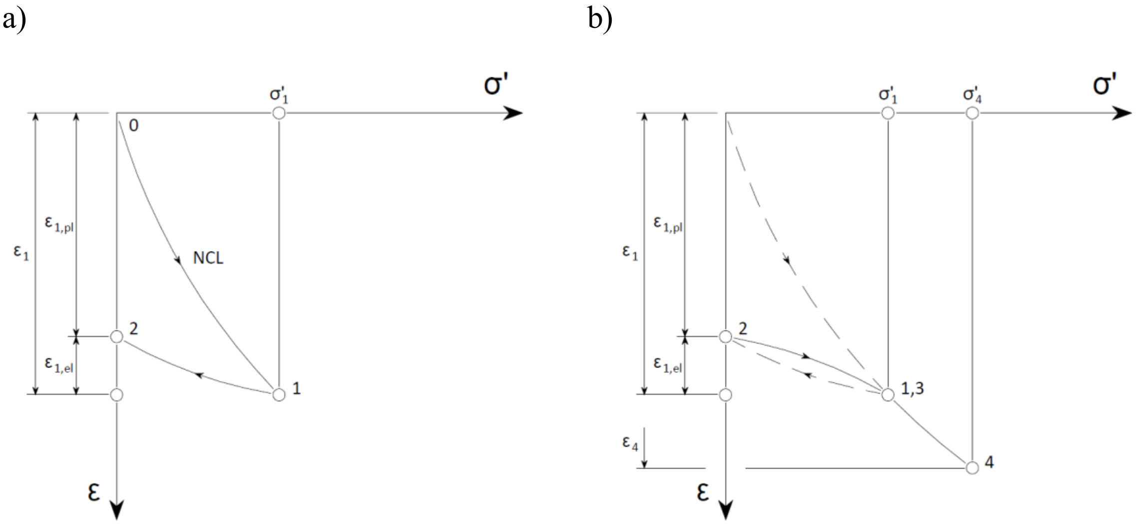


Figure 2-5: Consolidation states

In Figure 2-6, a one-dimensional consolidation analogy with no possibilities for lateral strain has been illustrated to easier understand the consolidation process of a water saturated soil under compression. Exemplifying the process, a cylinder filled with water has been used. The cylinder is equipped with a spring attached to a stiff plate that is completely in contact with the inner wall of the cylinder, because the water should not be able to penetrate. On the other hand, the plate is fitted with a valve that can be opened and closed, which may correspond to the permeability of the soil. The spring inside the cylinder can be compared to the soil skeleton (particles in the soil), while the water represents the pore water (Craig, 2004).

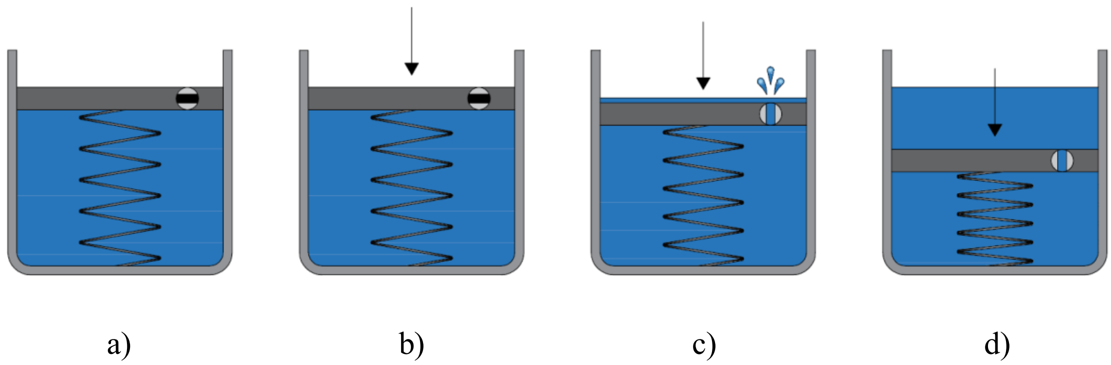


Figure 2-6: Consolidation analogy

In the initial stage when the plate is loaded, Figure 2-6 b), the incompressible water will completely carry the applied load as long as the valve is closed. The applied load on the plate corresponds to the total vertical stress in the soil. At this stage, no load is transferred to the spring i.e. the effective stress in the soil is unchanged. This represents an *undrained state* of a soil. Immediately when the valve opens, a water flow will occur, see Figure 2-6 c). The water drains, in other words, a reduction of excess pore water pressure takes place. It is also described as a water dissipation. In most cases, the pore water flow occurs in two or three dimensions to places with lower pore pressure such as the ground surface or adjacent drainage layers. This illustration, on the other hand, is only a one-dimensional consolidation process. Thus, the water flow only goes vertically. The water dissipation allows the plate to freely move as well as the spring can be compressed. In reality, this means that the particles are free to go to new places, resulting in an increase of the interparticle forces. As the spring is compressed and water dissipates, the load is increasingly transmitted to the spring as well as the pore volume is reduced (Craig, 2004).

In practice, the drainage proceeds until the pore water pressure again is in equilibrium with the value governed by the position of water level. This is something that varies a lot in real life depending on the permeability of the soil, a so-called hydrodynamic delay (Axelsson & Mattsson, 2016). For a coarse soil, it is something that can happen rapidly, while for a fine-grained soil can take up to weeks if not years. Anyway, after a certain time all excess pore water will be flowed out and no excess pore water pressure will exist anymore in the soil. The applied load has entirely been transferred to the spring (soil skeleton), which means that the effective stress is equal to total vertical stress, as shown in Figure 2-6 d). At this moment, the soil is in a *drained state* and consolidation is complete (Craig, 2004).

2.5 Shear in soil skeleton

Shear in a soil skeleton is said to occur when the soil mass starts to move tangentially along a shear plane. A deformation will usually occur under a shear in a soil skeleton, i.e. a volume change. The deformation varies depending on either the consolidation state of a fine-grained soil or the relative density of a coarse-grained soil. A normally consolidated fine-grained soil and a loosely densified coarse-grained soil that are exposed to shear decrease somewhat in volume under the action. The phenomenon means that a soil *contracts*. Figure 2-7 shows how the overlying particles in an ideal grainy soil mass move into the empty spaces between underlying particles. However, it can be said that for a long driven shear, in case of failure state when the compression strain increases, the volume will no longer decrease. The soil mass approaches a critical volume or, in other words, a critical state (Axelsson & Mattsson, 2016).

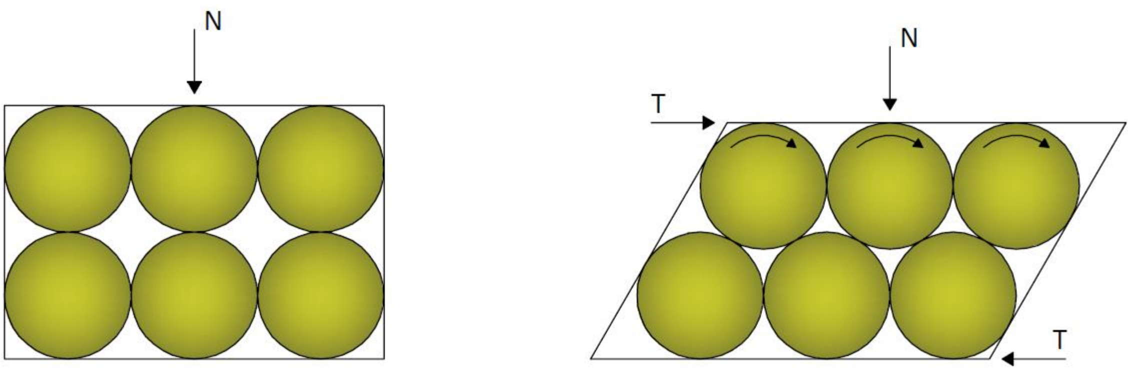


Figure 2-7: A normally consolidated soil contracts

On the other hand, a considerably overconsolidated fine-grained soil and a well densified coarse-grained soil tend to increase in the volume under shear. This has been demonstrated in Figure 2-8 for an ideal grainy soil mass. When the soil mass, being in an already dense state, are pushed in sideways the overlying particles will be lifted over the underlying particles. In such way the volume increases. This is used to be named as *dilatancy*. The soil mass is going to be compressed in the early stage of the shear. However, after a long driven shear, the volume is going to increase and approach a limit in the void ratio. The soil mass is said to be in a critical state (Axelsson & Mattsson, 2016).

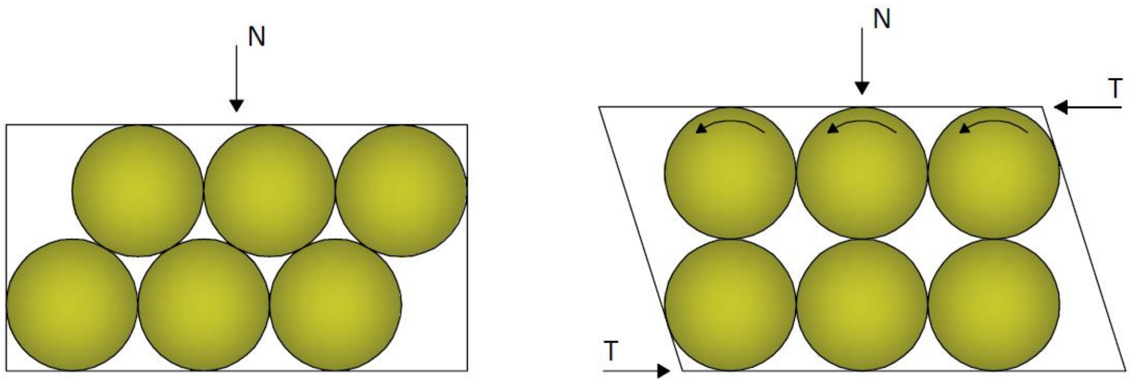


Figure 2-8: Shear under dilatancy

These types of shear clearly prove an irreversible (plastic) deformation. If the compressive forces stop acting, the volume reduction will remain the same. However, the shear presents also an elastoplastic deformation as compression. Since a part of the shear strain returns to its original shape (elastic) at an unloading, but still the greater part of the strain remains plastic (Axelsson & Mattsson, 2016).

In order to avoid the occurrence of a shear failure, it is required that shear strength is greater than the shear stress. The shear strength is derived from friction and cohesion in a soil. The interparticle contact is of great importance and furthermore also the effective normal stress (Craig, 2004). Friction, which appears in coarse-grained soils, depends largely on the grain size, grain distribution, shape of particles and void ratio. Cohesion, on the other hand, acts among fine-grained soils such as clays and to some extent silts. The particles in a cohesive soil are surrounded of a highly viscous film of bound water. The bonds occur via these water films due to attractive bonding forces, which may happen at a sufficiently close distance (Axelsson & Mattsson, 2016). The shear strength is determined by using Coulomb's equation (2.19). The parameters are expressed in term of effective values as only the soil particles can withstand shear. Figure 2-9 presents the parameters and the failure envelope of Mohr-Coulomb failure criterion. Shear failure is going to occur when shear stress acts above the failure envelope (Craig, 2004).

$$\tau_f = c' + \sigma_f{}' \cdot tan\phi' \tag{2.19}$$

where:

τ_f = shear strength

c' = cohesion

$\sigma_f{}'$ = effective normal stress at failure

ϕ' = angle of shearing resistance

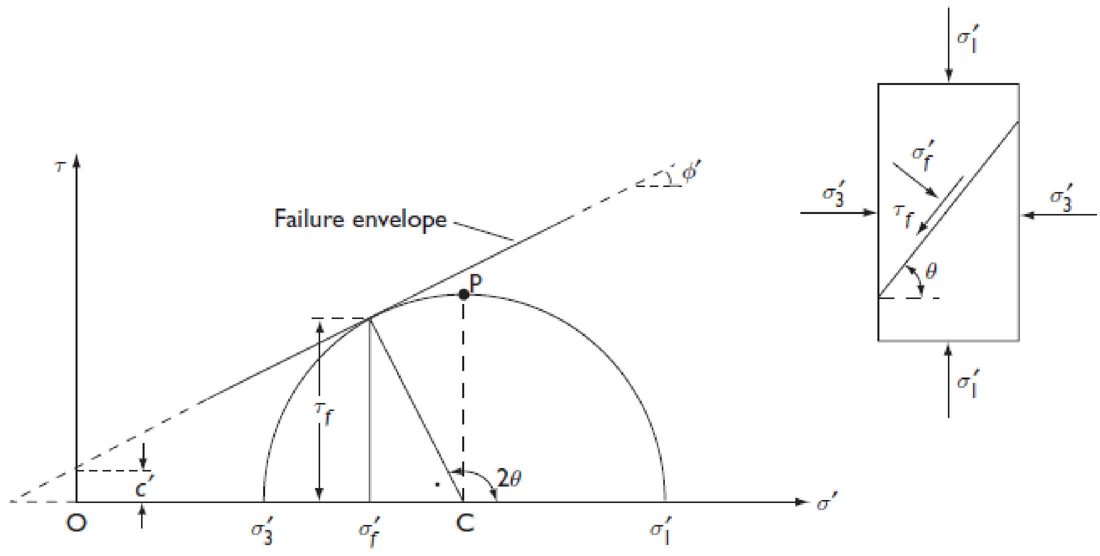


Figure 2-9: Mohr-Coulomb failure criterion for a soil with friction and cohesion [2]

Since the above-mentioned Coulomb's equation is applied for the drained shear strength, the undrained shear strength will somewhat differ. A very fine-grained soil may not allowed the consolidation to take place at that time the total normal stress is increased. There won't be any changes in the effective normal stress and only the pore water pressure will increase. An undrained condition occurs. As no effective normal stress acts in the soil skeleton the shearing resistance will then also be zero. The undrained shear strength τ_{fu} will be equal to the undrained cohesion c_u, which can be seen in Figure 2-10. Regardless of the applied stress, failures will always occur at the same principal stress difference (or deviator stress, q_{fu}). This shows that the shear resistance of a cohesive soil isn't dependent on the applied stress (Axelsson & Mattsson, 2016).

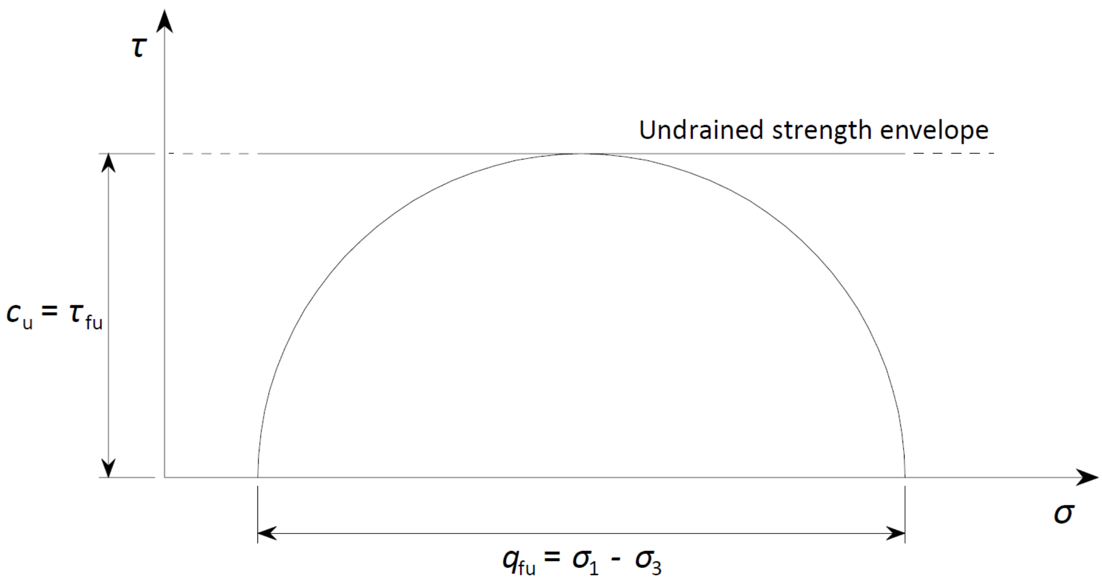


Figure 2-10: Undrained strength envelope

3 Soil Behaviour

A soil exposed to external loads usually behaves critically against its functionality in situations where soil improvements have not been properly performed. It will be clarified in more detail in this chapter about the behaviour of a soil. The three most significant forms will be; *settlement*, *bearing capacity* and *liquefaction*. If any of these do not keep the criteria, devastating consequences can occur. Therefore, measures should be taken to mitigate or completely prevent geotechnical actions such as these by executing alternative soil improvement methods (Axelsson & Mattsson, 2016).

3.1 Settlement

The settlement of a soil is divided into several different compression stages, which can be a long period of time (Craig, 2004). The settlement itself can be originated from many mechanisms, such as compaction, consolidation, water movement and creep (Whitlow, 2001).

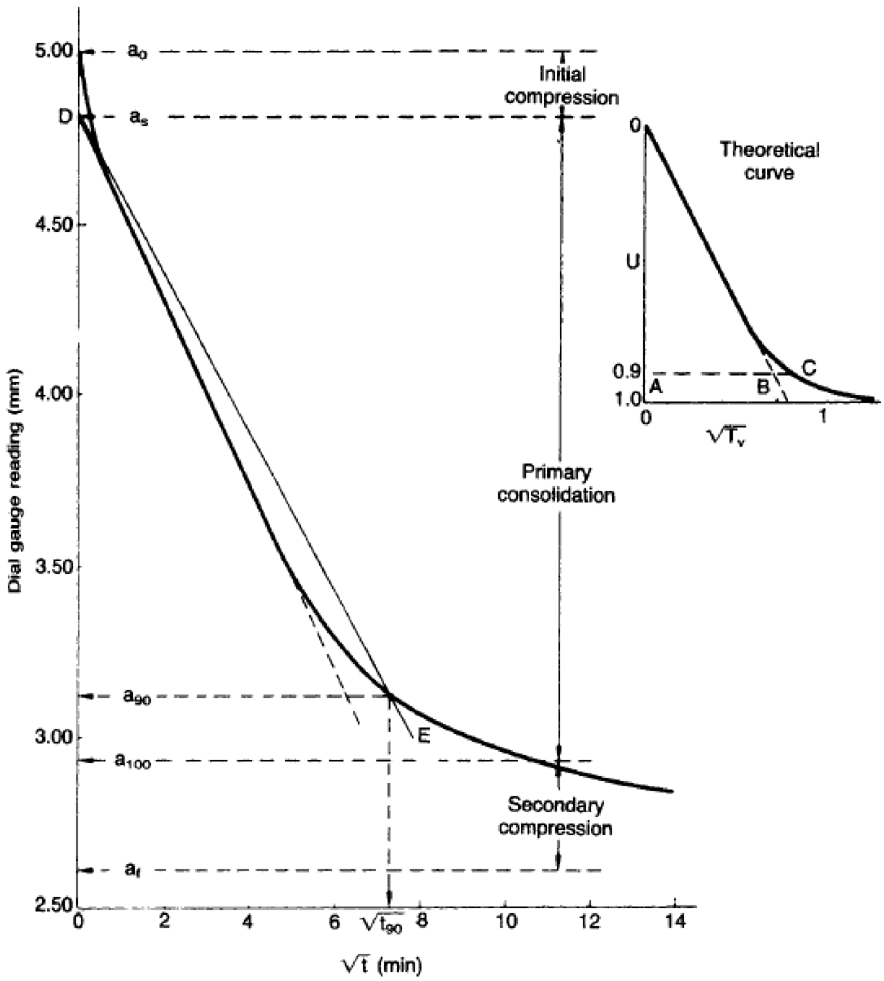


Figure 3-1: Compression stages [3]

Figure 3-1 demonstrates each stages of compression related to the time. The first step called *initial compression* is mostly generated by the compression of small proportion of air presented in the soil. Further the *primary consolidation* takes place, which includes the process of consolidation explained in the chapter 2.4. There the permeability of soil will completely control the time of process. When the dissipation of excess pore water pressure has ceased, a *secondary compression* will exist followed by a creep in the soil skeleton. This creep is particularly common in clayey soil where the particles are surrounded by a highly viscous film of absorbed water. The particles start to move closer to each other as the absorbed water viscously flows from the film around the solid particles (Craig, 2004).

3.2 Bearing Capacity

There are two different types of foundations; shallow foundation and deep foundation. The latter involves foundation on piles, which intend to transfer the load to deeper layers such as suitable soil or solid bottom (rock). Thus, it is not addressed in this context. By contrast, the shallow foundation covers foundations as footings and rafts, which are possible to be used directly on soils, providing that the capacity is great enough to support the structural loads. It assumes that the bearing capacity of soil is greater than the induced pressure to the foundation. Otherwise, shear failure occurs in the supporting soil. However, it is not only soil properties that determine bearing capacity but also the shape and size of the foundation (Craig, 2004; Axelsson & Mattsson, 2016).

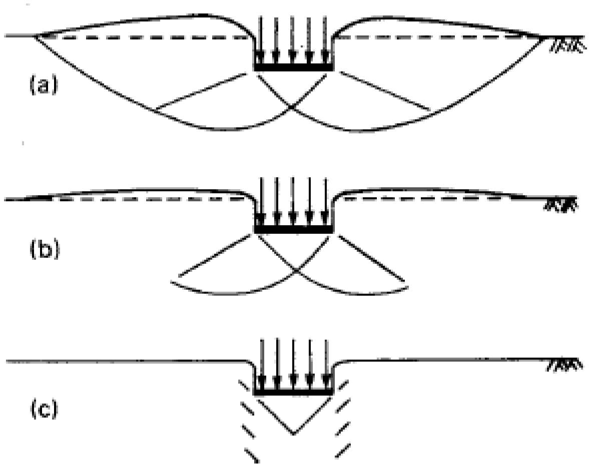


Figure 3-2: Shear failure modes [4]

Figure 3.2 shows the different shear failure modes that are going to appear if the underlying soil near the surface is insufficient to support the structural loads. Accordingly the ultimate bearing capacity is defined. Mode a) presents a *general shear failure* occurring between the edges of the footing and the ground surface. A plastic equilibrium of the soil is attained around the edges of the footing due to the increased pressure, which will further propagate downwards and outwards. A fully developed plastic equilibrium is finally found above the failure surface before a final slip movement happens. This usually arises in dense and stiff soils. Mode b) is kind of similar to pervious mode, but the plastic equilibrium is only partially developed. The failure surfaces will not extent all the way to the ground surface and the heaving is slightly smaller than in mode a). This is called *local shear failure* and is seen mostly for high compressible soils. The last failure mode, *punching shear failure*, is a simultaneously occurrence of high compression and laterally shear around the edges of the footing. The mode develops comparatively large settlements in low compressible soils, when the foundation is placed at a significant depth. In general, the failure modes are dependent on the compressibility of the soil as well as the foundation depth in relation to the width (Craig, 2004).

3.3 Liquefaction

A soil affected by a sudden change in the pore water pressure will have a fast drop in the shear strength and become instantly liquid. This phenomenon is called liquefaction and will occur mainly in loose saturated fine-grained soils under the influence of dynamic forces (cyclic loading); i.e. similarly occurring during earthquakes or by vibrations from machinery. This cyclic loading causes a compaction (volume reduction) of the soil. There the loose sand contracts and an increase in the pore water pressure occurs. Since there is not enough time for the pore water to dissipate it will result in an undrained condition. The effective stress will become zero and the interparticle forces will no longer exist in the soil skeleton. The sand is in a liquid state with an insignificant shear strength (Craig, 2004; Möller, Larsson, Bengtsson, & Moritz, 2000). In the section 2.5 an illustration (Figure 2-7) is made similar to the action of soil particles during a cyclic loading.

Liquefaction may appear where the combination of in-situ density and cyclic deformation is critical. This means when the void ratio is high and the confining pressure is low the liquefaction will more easily happen. Also if the produced strains from the cyclic loading are very large it does not require that much number of cycles for liquefaction to happen (Craig, 2004).

4 Impact Oriented Compaction

4.1 General

Dynamic compaction is a soil improvement method based on strengthening weak soils by high-energy impact. The kinetic energy of the tamper is transferred to the deeper soil layers, which leads to densification of soil. The method intends to increase the bearing capacity, reduce the settlements as well as mitigate the liquefaction (BAUER Maschinen: Process, Bauer Dynamic Compaction (BDC), 2017; Slocombe, 2005).

There are several types of impact oriented compaction methods, but as mentioned earlier, this paper will only cover the conventional dynamic compaction method and the more recently developed rapid impact compaction method. These kinds of soil improvement method are very efficient in their way of working, given that they are able to densify a large deposit of soil in one phase instead of dividing the deposit into several thinner layers (Cofra: Documents, CDC compaction).

4.2 Soil Improvement Methods

4.2.1 Dynamic Compaction

Dynamic compaction (DC) is a quite traditional and old-fashioned method, which has its origins far back in time. Drawings ever since the early Chinese era are found there the technique reveals; also the Romans used the method of their construction. Since then, it has also occurred in several projects both during the 19th century and the early part of the twentieth century. However, it was first in 1970 in France that the method became more frequently used when the cranes became larger. This resulted in the method receiving a much higher energy tamping level improving the soil deeper down, thereof also the name *deep dynamic compaction* that is very common among this soil improvement technique. A few years later, the method also appeared in the UK and North America (Slocombe, 2005).

The principle of DC is to improve the soils by repeated dropping of a heavy weight (pounder or tamper) onto the ground surface from a predetermined height. Thus, dynamic compaction only requires a crawler crane with enough capacity and a free-falling weight of an appropriate size (see Figure 4-1). Therefore, as dynamic compaction only requires rather basic equipment, potentially with minor modifications due to the safety requirements, most contractors can perform it (Slocombe, 2005). The weights can usually range from 5 to 30 tons (about 50 to 300 kN) and is typically dropped from heights between 10 to 30 meters (Lukas, 1995). Even larger equipment has appeared at larger sites reaching greater depths (Slocombe, 2005). This provides dynamic compaction with an energy per drop between 500 to 9,000 kNm, with a frequency of one to two drops per minute. Thus, the power (energy per time unit) varies between 0.6 to 18 MNm/min.

Figure 4-1: A crawler crane equipped with a free-falling weight [5]

The actual use of the weights as well as the dropping height can differ a bit depending largely on the soil condition, but the weight should at least be made of a durable material like steel or concrete. The shape, in turn, can be of a square or circular character (BAUER Maschinen: Process, Bauer Dynamic Compaction (BDC), 2017). In non-cohesive or granular soils a tamper with smaller base area is usually seen, while for cohesive soils a wider base area is better suited (Akhtar, 2018). It is very common to use a narrower weight in cases where materials want to be driven deeper down into a softer soil by creating a sort of column. Also, the choice of weight and height has its role in the resulting degree of compaction (Slocombe, 2005).

The DC improvement of the soil is typically divided into three different stages or tamping passes with compaction points placed in a grid pattern, see Figure 4-2. The first tamping pass, called *primary treatment*, intends to reach the deepest layer, as shown in Figure 4-2 a). Thereof the DC treatment is usually called a "bottom-up" process. The treatment is done by implementing a quite wide grid pattern and dropping a large weight from a high elevation. Then the second pass, which is named as *secondary treatment*, is aiming for the middle layer introducing a grid pattern in between the first pattern. Figure 4-2 b) displays the primary grid P as yellow circles and the secondary grid S as red squares. In the secondary treatment both the number and height of the drops are reduced. The last pass, also called *final ironing pass*, is treating the surface layer by a small number of drops from low height over the whole actual area (Slocombe, 2005). After each performed pass, the developed craters are either levelled or filled with some granular fill before the next pass takes place (Lukas, 1995).

a) b)

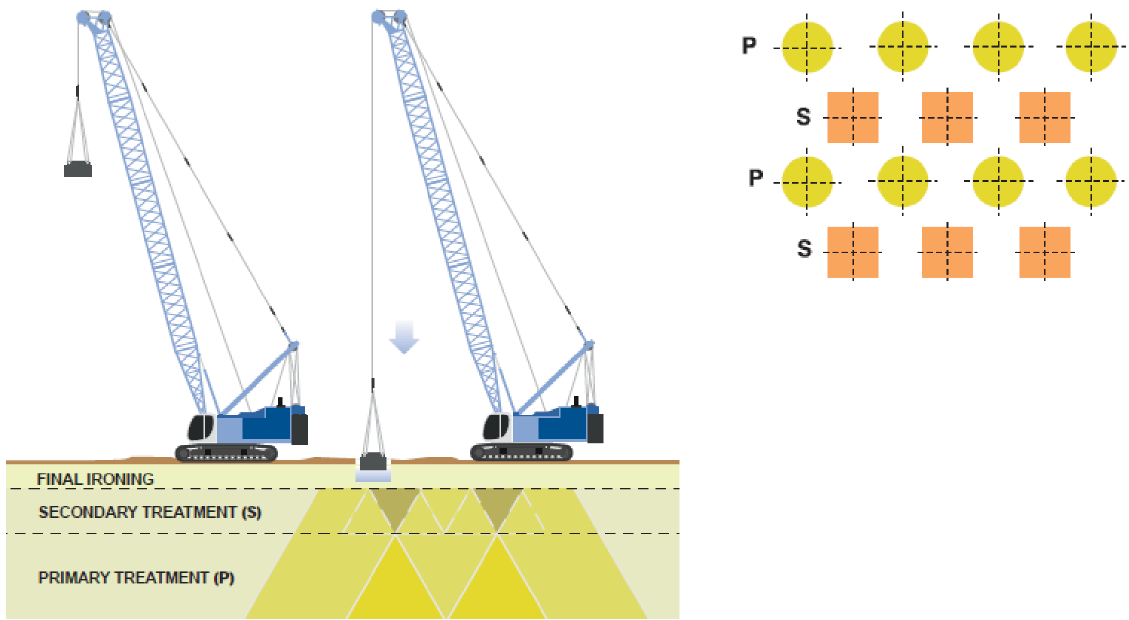


Figure 4-2: a) The three tamping passes b) Typical grid pattern [6]

The actual mechanism of densifying a soil, works in such a way that the forces, starting from the ground surface, forcing the soil particles into a movement. The friction between the soil particles ceases to work. This movement occurs due to shock waves transmitted through the soil, when the weight is dropped onto the ground surface. These shock waves may exist as either volumetric waves or boundary waves depending on the direction they move or the influence on soil they have (Möller, Larsson, Bengtsson, & Moritz, 2000; Tan, 2007).

Volumetric waves appear as compression wave or shear wave, i.e. vertical wave or horizontal wave, and are something that causes the particles to be rearranged and compacted. The compression waves and shear waves make it possible to expel the water through so-called cracks in soil mass. In this way, DC treatment allows a rapid dissipation of the pore water pressure. The soil can then consolidate and the soil particles are able to move into a denser state. A compaction process of a saturated soil is also usually called *dynamic consolidation* because of this. On the other hand, the boundary waves move along the ground surface. These waves will no longer densify the soil particles but only loosen the ground surface. For that reason, the ground should be treated with the final ironing pass. Even the shear waves make the ground surface loose (Tan, 2007).

4.2.2 Rapid Impact Compaction

Rapid impact compaction (RIC) is a rather new form of soil improvement technique. The device was primarily developed in the early 90’s by the British company *BSP International Foundation Ltd* in association with the British Military to quickly repair bomb craters on aircraft runaways (BSP International Foundations: What is RIC?, n.d.). The civilian use of the device has become more common during the past, but still the frequent use is quite rare in the Nordic countries (Havukainen, 2014).

In the same manner as DC, a falling weight is utilized to compact the ground, but the approach of RIC is still fundamentally different. A hydraulic piling hammer strikes a patented foot from a controlled height. The foot rests all the time on the ground during the operation, which causes no risk of flying debris to occur at the site as well as a more efficiently applied energy when compacting (Serridge & Synac, 2006). The diameter of the impact foot ranges generally from 1.0 to 2.0 m, but even greater diameters are found (BSP International Foundations: RIC Models, n.d.; Havukainen, 2014). All this is assembled to a hammer rig, which in turn is mounted on an excavator (Figure 4.3). It can also be controlled via a crane, but it causes rather strict height limit (Adam & Paulmichl, 2007).

Figure 4-3: Rapid impact compactor on a track-mounted excavator [7]

The falling weight consists of a mass between 5 to 16 ton (about 50 to 160 kN), which is generally dropped from a height of 1.2 meter. This delivers a maximum energy per blow between 60 to 192 kNm, which is much lower than DC. RIC has on the other hand an operation rate of 40-60 blows per minute, which imparts a maximum energy per minute that varies between 2.4 to 11.5 MNm/min (BSP International Foundations: RIC Models, n.d.; Havukainen, 2014). This results in that the total energy input per time unit is quite the same for the two methods.

The first few blows in the RIC treatment create a dense plug of soil instantly underneath the impact foot (Watts & Cooper, 2010). A plastic deformation occurs and the applied energy will be absorbed in the soil. As the soil is densified, it will become more elastic when the properties of the soil change. The occurring elasticity makes it possible for the applied energy to propagate to even greater depths (Möller, Larsson, Bengtsson, & Moritz, 2000). This makes the RIC treatment known as a "top-down" process. During further processing the blows force this plug successively deeper down, which also compacts the soil in deeper layers. In this way, the process continues further as long as a little penetration of the impact foot can be achieved with an increase of blows. The effect of the compaction will largely occur vertically below the compaction point. Therefore the treatment should be carried out within a square or triangular grid pattern with an acceptable centre distance, or in an order of an arc pattern with the base carrier for the RIC equipment as the centre of rotation. Because one should ensure that an effective and a homogeneous treatment is performed, the secondary passes are shifted slightly in relation to primary passes so that they are positioned in between them (Watts & Cooper, 2010; Serridge & Synac, 2006). In Figure 4-4 to Figure 4-6 the different grid patterns can be studied.

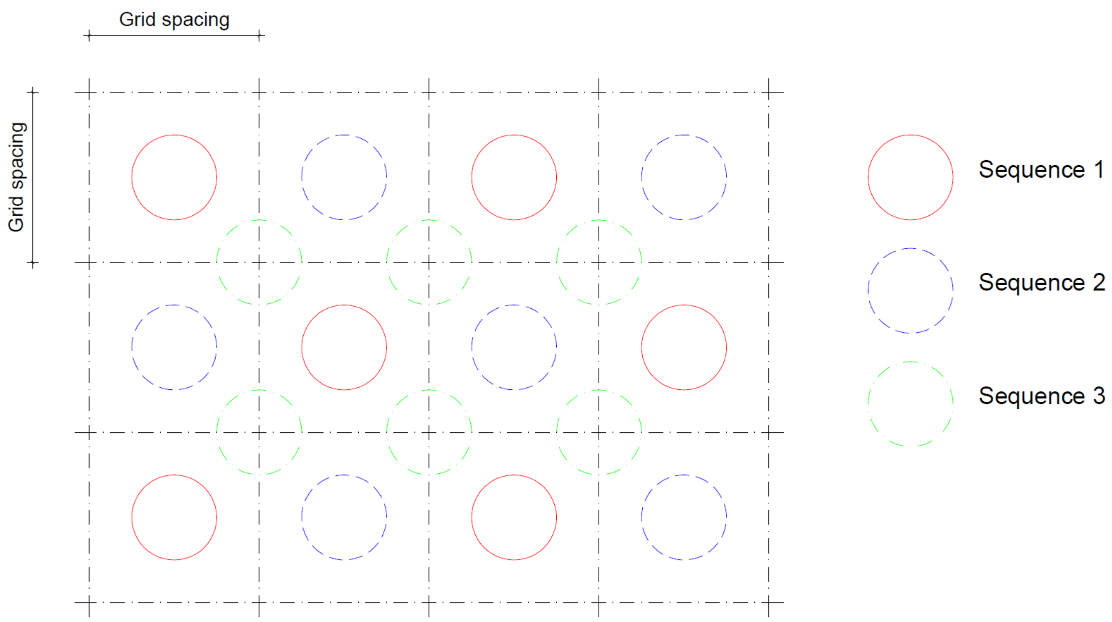
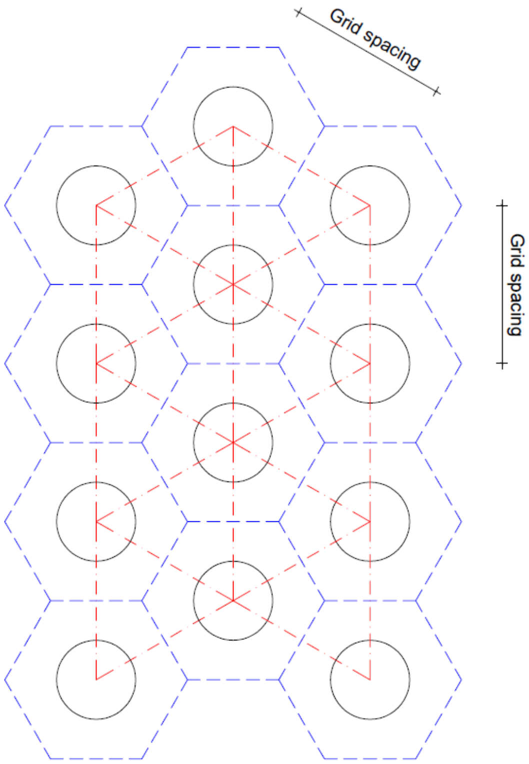


Figure 4-4: Square pattern



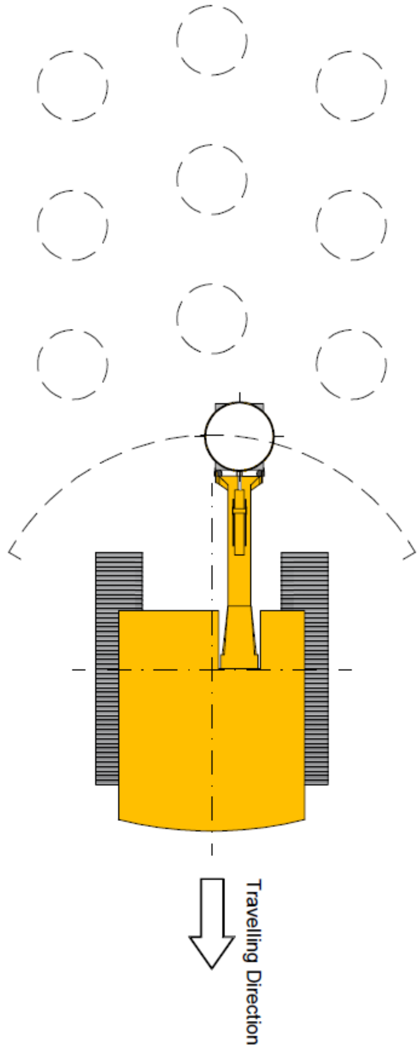


Figure 4-5: Triangular pattern

Figure 4-6: Arc pattern with the base carrier as the centre of rotation

4.3 Suitability

Impact oriented compaction demands some requirements of the soil conditions considering the suitability for the performance. An insufficient soil will make it difficult for dynamic compaction to perform a proper compaction. The soils being compacted at most of the sites are below the water table or, in other words, fully saturated. The deposit should thus be of a relatively pervious soil so the excess pore water has the ability to dissipate quickly and then allow the soil particles to transfer into a denser state of compaction. The densification by using an impact oriented compaction method works most efficiently when the permeability is high and the drainage is well-functioning, which generates a low degree of saturation (Lukas, 1995). In the Figure 4-7 a soil gradation curve has been displayed showing the range for the most favourable soil deposits and vice versa.

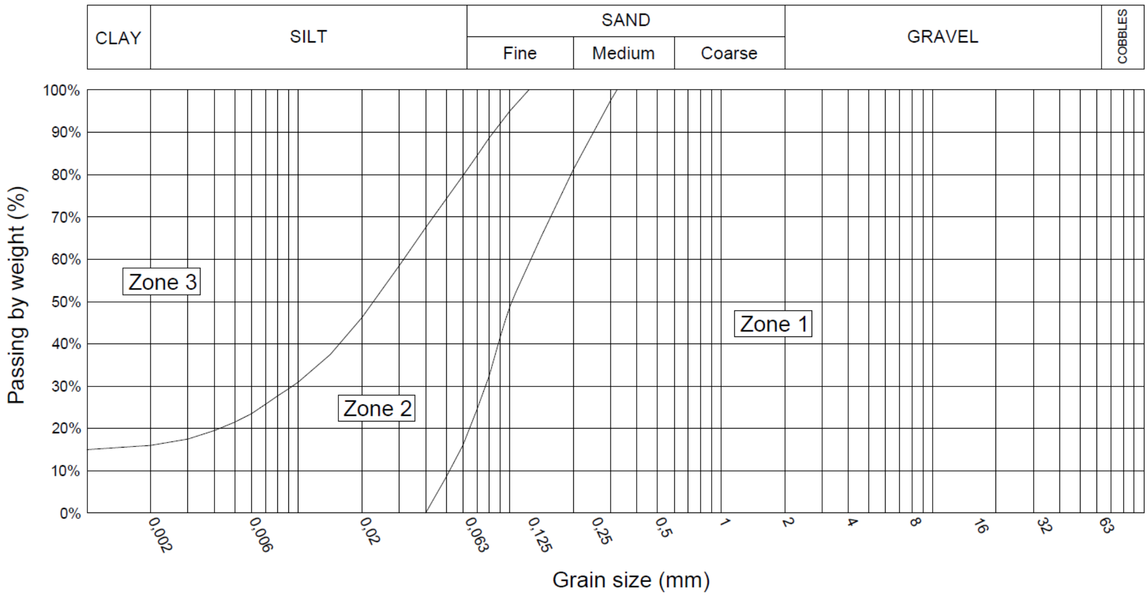


Figure 4-7: Range of soil gradation for the suitability of dynamic compaction [8]

According to the above-mentioned graph, the most suitable type of deposit for dynamic compaction is found in the far-right section of the graph, *Zone 1*, including pervious granular soils. As long as these deposits are above the ground water level, the soil particles are pushed into a denser state immediately and an excellent engineering performance is achieved. What may happen is that for coarse-fill materials a hard 'plug' is created preventing further penetration of stress impulses to the deeper layers. Something that is more undesirable in DC treatment compared to RIC treatment, because the DC process strives to densify the soil mass from bottom-up using shock waves. On the other hand, in cases where the pervious deposits are situated below the ground water level, the high amount of dynamic impulses cause an excess pore water pressure that rises to the surface level inducing liquefaction. For a highly permeable soil the pore water pressure caused by the impact of pounder will dissipate almost directly. The actual density and grading of the soils are the leading factors considering the speed. Pervious deposits need not only consist of natural sands and gravels that can also occur in the form of building rubble, construction debris, industrial waste fills, etc. (Lukas, 1995; Slocombe, 2005).

Deposits not suitable for dynamic compaction are situated on the left side of the graph, *Zone 3*, refer to Figure 4-7. This part of soil gradation includes clayey soils consisting only of fines, a so-called cohesion soil. The particles for these soils are surrounded by highly viscous film of absorbed water, which was also mentioned in the chapter 2.5. As the pore water cannot transfer any shear forces, it will be more difficult for the soil particles to be rearranged. The only way to perform soil improvement for these types of deposit is to reduce water content. The excess pore water pressure can't dissipate that properly during these conditions due to the low permeability except perhaps over a long-term period. This is something that poses problems for implementing a proper improvement for both DC and RIC, which also makes it neither profitable nor sufficient. An improvement in clayey fill deposits assumes that the soil mass is only partially saturated, as it were situated far above the ground water level and with a good drainage at the surface. The water content of clayey soils should not be greater than the plastic limit of the deposit in order to perform a good dynamic compaction (Lukas, 1995).

In between these two soil categories is an intermediate soil deposit category including silts, clayey silts and sandy silts. These soils are found in *Zone 2* in Figure 4-7. Dynamic compaction acts pretty well for these deposits too, but due to the lower permeability it assumes that the energy is applied by increasing the number of phases or passes. There must also be sufficiently long recovery between the phases or passes, because the time that it takes for the excess pore water to dissipate may last for a long time during these conditions. There are cases where it has lasted for several days and even weeks (Lukas, 1995; Slocombe, 2005).

4.4 Treatment Depth and Design

The depth of improvement differs somewhat between DC and RIC, but in general the upcoming range of depth for both soil improvement methods is based on the same factors. First of all, the initial condition of the existing soil has a major influence on the determination of the treatment depth. An already very dense layer does not affect the treatment depth that much compared to a soil in a looser state. Furthermore, the soil type and the energy input play also a big role in the depth of improvement (Slocombe, 2005).

The soil type is the most unpredictable factor of these, and has a significant impact on whether the method can be implemented or not. There is a huge difference in ease of densify a non-cohesive soil and a cohesive soil, if the soil contains a high amount of fines the depth of effectiveness will be reduced. The same goes for the position of the groundwater level as well as the stratigraphy of the subsurface. These are factors that reduce the effective energy transferred down to the soil deposit and inhibit the improvement to depth (Slocombe, 2005; Serridge & Synac, 2006).

Something that has a major impact on the depth of RIC is the total energy input. The energy per blow for RIC is of a very small character compared to DC. Therefore, the number of blows at a compaction location point or the applied energy across the area will have a much greater importance to the effective depth of RIC treatment. This originates from the progressive top down improvement of the treated ground (Serridge & Synac, 2006). Apart from the RIC, the effective depth of DC treatment is more easily predicted and relates directly to the metric energy input. Menard and Broise (1976) were the first to develop an equation in order to estimate DC treatment depth, but later modified by other actors for instance Mayne, Jones and Dumas (1984), see equation (4.1).

$$D_i = n_c \sqrt{W_t H_d} \qquad (4.1)$$

where:

D_i = depth of improvement in meters

W_t = mass of drop weight (tamper) in metric tons

H_d = drop height in meters

n_c = empirical coefficient

The empirical coefficient n_c that has been produced via trials done on the sites, is an influential factor in the sense of depth of improvement. The coefficients for different soil types are shown in the Table 4-1.

Table 4-1: Recommended n_c value for different soil types [2]

Soil Type	Degree of Saturation	n_c*
Pervious soil deposits – granular soils	High	0.5
	Low	0.5-0.6
Semi-pervious deposits – primarily silts with $I_P < 8$	High	0.35-0.4
	Low	0.4-0.5
Impervious deposits – primarily clayey soils with $I_P >$ 8	High	Not recommended
	Low ($w < w_P$)	0.35-0.4

I_P = plasticity index, w = moisture content and w_P = plastic limit.

* For $W_t H_d$ = 1-3 MJ/m^2 and a tamper drop using a single cable.

The coefficient is based on a lot of different parameters that adversely affect the depth of improvement. Primarily, the soil type has a major impact on the result. There are soils such as granular deposits that get a relatively good depth of improvement even after a few drops, while then there are those soils that need a larger number of drops before an acceptable result can be seen. This means that the total applied energy has some influence on the treatment depth too. In addition to these two parameters, the drop mechanism is also one of the major impacts for the depth of improvement. Lifting and dropping a weight with a single cable attached is more energy consuming compared to a weight that is raised and then allowed to free fall. Although the energy loss is less, the use of process is rarely seen. Since every lift requires some lifting mechanism to be attached, the process becomes much more time consuming compared to a weight attached to a single cable all the time (Lukas, 1995).

These three above-mentioned variables are the most discussed and occurring factors that are included in the correlation coefficient n_c. However, there are parameters that are more unpredictable, and therefore should be determined more on a case by case basis. These may be the presence of energy absorbing layers such as hard or soft layers that prevent the energy from transferring to greater depths. Alternatively, the contact pressure developed via the tamper could as well cause disturbance in the depth of improvement and should therefore be kept within a good range, neither too low nor too high (Lukas, 1995).

Regarding the treatment depth accomplished with RIC Oshima & Takada (1997) demonstrated in a nearly dry sand sample that the use of total cumulative energy and total cumulative momentum of the compactor are a better sign in the sense of estimating the degree of improvement. Thus, better in predicting the depth of improvement and change in the relative density within RIC treatment. They conclude further that a use of total momentum gives a more favourable result. This was later also confirmed by Lee & Gu (2004). The way of calculating the degree of compaction is shown in the equations (4.2), (4.3) and (4.4), starting with momentum calculation produced by Mayne and Jones (1983).

$$v = \sqrt{2gH_d} \tag{4.2}$$

where:

v = impact velocity

g = gravitational acceleration

H_d = drop height

$$M = W_t \cdot v \qquad (4.3)$$

where:

M = momentum

W_t = weight of tamper

v = impact velocity

Oshima & Takada (1997) presented in their paper that the depth of improvement is proportional to the logarithm of cumulative momentum, see the equation (4.4).

$$D_i = a_z + b_z \log(N_d \cdot M) \qquad (4.4)$$

where:

D_i = depth of improvement

a_Z and b_Z = depth correlation coefficients

N_d = number of drops

M = momentum

Table 4-2 displays the depth correlation coefficients according to the differential change in relative density. These coefficients have been calculated from the chart (Fig. 10) in paper (Oshima & Takada, 1997, p. 1644), see *Appendix 1*. In the same way as equation (4.4) the radius of improvement zone can be calculated. Then, instead of depth correlation coefficients the radial correlation coefficients are used, see *Appendix 2*.

Table 4-2: Depth correlation coefficients

Depth Coefficients:	a_Z	b_Z
ΔD_r = 40 %	-7.509	3.301
ΔD_r = 20 %	-8.691	3.94
ΔD_r = 10 %	-9.933	4.586

The treatment depths among the two soil improvement methods have been shown in different graphs below. Figure 4-8 presents an illustration of the range of treatment depth using DC. There x-axis represents the energy input and y-axis represents the depth of influence. Then the depth is going to change depending on the soil type, which is directly connected to the inclination of the line. It is controlled by a coefficient (empirical coefficient n_c). It can also be mentioned that the treatment depth varies with initial strength, which is displayed in the graph as different zones (Slocombe, 2005).

Figure 4-9 introduces the range of treatment depth using RIC. The x-axis has in this case got the variable, number of blows, while the y-axis stays as depth of influence. This graph shows a depth of influence for three different zones according to the change in relative density. There is an increase by 10, 20 and 40 percentage. Each zone covers a treatment depth produced by a variation of hammers falling from a height of 1.2 m. For the deepest achieved depths a 16-ton hammer is used and for the shallowest achieved depths a 5-ton hammer is used. Then, in between, it can be hammers of the size 7, 9 and 12 ton.

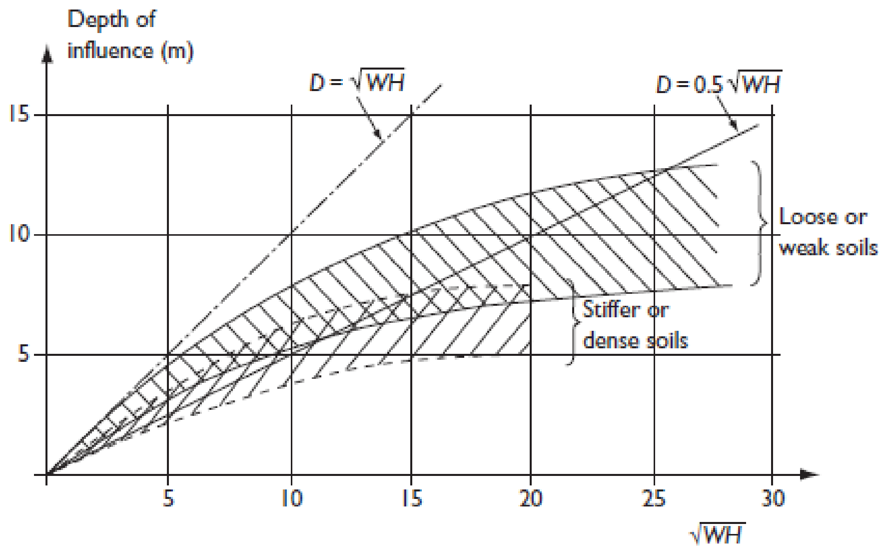


Figure 4-8: Depth of treatment using DC [9]

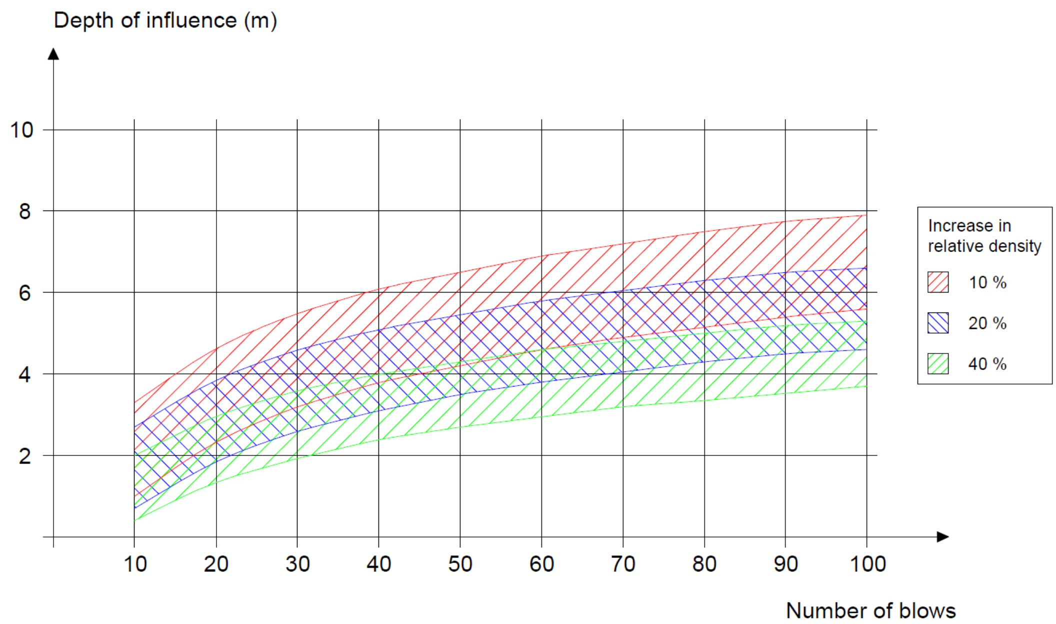


Figure 4-9: Depth of treatment using RIC

4.5 Testing and Quality Control

Within any major DC and RIC works there are compaction trials implemented at the initial stage to evaluate the ground response as well as to perform the quality control. On-going monitoring and testing are utilized to ensure that an appropriate amount of energy has been transferred to the soil deposit and the performance requirements are going to be fulfilled (Becker, 2011; Serridge & Synac, 2006).

The compaction units are equipped with a GPS guided crane monitoring system, which is a so-called data acquisition system, to observe the compaction process for the quality control. This records the performance of either the hammer in RIC or the pounder in DC as well as the penetration rate of improvement. Based on the observations a stopping criterion can be defined by either the total number of blows, the induced settlement (average settlement per blow of the last three blows) or the total settlement (Vink & Dijkstra, 2016; BAUER Maschinen: Process, Bauer Dynamic Compaction (BDC), 2017). The on-board monitor alerts directly to the driver then when the stopping criteria is met at the current impact point, and the unit is ready to move to the next compaction location (Becker, 2011).

Beyond that, quality assurance testing is executed by penetration tests before and after the trials as well as during the actual compaction works. Using the penetration tests a comparison of pre- and post-improvement tests can be presented to verify the degree of compaction. There are a handful of different penetration testing methods such as dynamic penetrometer tests (DPT), cone penetration tests (CPT) and standard penetration tests (SPT) (Serridge & Synac, 2006; Becker, 2011). Additionally, pressuremeter test (PMT) and plate load tests are also sometimes used (Slocombe, 2005; Han, 2015).

The specified degree of compaction is consequently achieved by applying a sufficient amount of energy into the soil. Therefore, it is important to determine all the influencing parameters to reach the target, while deciding the stopping criteria, as a part in the design phase. The applied energy is usually calculated as an average for the entire area, see equation (4.5) (Lukas, 1995; Becker, 2011).

$$AE = \frac{N_d \cdot W_t \cdot H_d \cdot P}{A_e} \tag{4.5}$$

where:

AE = applied energy at each drop point

N_d = number of drops by one pass at each drop location

W_t = weight of tamper

H_d = drop height

P = number of passes

A_e = influence area of each impact point

($A_e = s^2$ for a square pattern or $0.867\ s^2$ for an equilateral triangular pattern)

s = drop spacing

4.6 Vibrations and Environmental Considerations

The ground vibrations should always be considered when using impact oriented compaction devices. It requires some caution especially in urban environments. Demands are made on these devices working in those areas due to potential damages to nearby buildings and buried utility lines as well as annoyance to human beings in general. The design criteria for buildings and structures as well as annoyance level for humans are usually defined by peak particle velocity, but the damages are also proved to be dependent upon the vibration frequencies. Therefore, it is assumed that careful monitoring of ground vibration levels and vibration frequencies are carried out to predict the values in beforehand (Lauzon, Morel, Briet, & Beaton, 2011; Slocombe, 2005; Becker, 2011).

Mayne, Jones & Dumas (1984) set the basis for calculating the peak particle velocity in the same way as for blasting, see equation (4.6). They concluded also that measurements of peak particle velocity are used to increase in the same manner as the number of blows increases at a compaction location. The reason is the soil becomes denser as compaction progresses. This also assumes that the cumulative compaction energy is considered when estimating peak particle velocity (Slocombe, 2005; Lauzon, Morel, Briet, & Beaton, 2011).

$$PPV\ (cm/s) \leq 7 \left(\frac{\sqrt{W_t\ H_d}}{d}\right)^{1.4} \qquad (4.6)$$

where:

PPV = peak particle velocity in cm/s

W_t = weight of tamper

H_d = drop height

d = distance from impact point

A year later, Mayne (1985) presented a new approach for the peak particle velocity that suited better for the set of his data. The impact velocity and the distance by the radius of the tamper were considered to normalize peak particle velocity. The approach is shown in equation (4.7), which reveals that RIC generated vibrations should be smaller than those from DC. Because of a smaller drop height together with a smaller diameter of the tamper (Lauzon, Morel, Briet, & Beaton, 2011).

$$PPV = 0.2\sqrt{2\ g\ H_d}\left(\frac{d}{r_o}\right)^{-1.7} \qquad (4.7)$$

where:

PPV = peak particle velocity

g = gravitational acceleration

H_d = drop height

d = distance from impact point

r_o = radius of tamper

Regarding the vibration frequency induced by DC it is typically in the range of 6 to 10 Hz (Lukas, 1995), while for RIC is a few times higher. Lauzon et al. (2011) show cases where RIC induced vibration frequency has achieved a level around 20 and 30 Hz and even up to 50 Hz depending a lot on the soil type and pre-improvement density. This demonstrates that the vibration frequency induced by RIC is of a higher level than the primary frequency of a structure. Thus, using RIC equipment makes it easier working closer to the actual structures because of less vibration amplification expected and since the safe level of vibrations tend to increase with a growing frequency (Lauzon, Morel, Briet, & Beaton, 2011).

Methods should be applied to reduce the effect of ground vibrations in cases where they are problematic for the work to be performed. The simplest way is to reduce the drop height and instead insert a larger number of drops per impact point, or use a lighter weight. These ways make it more difficult to reach the deeper layers. Therefore, open trenches to a sufficient depth need to be excavated to cut off the surface waves (Lukas, 1995; Slocombe, 2005).

Air-borne noise levels must also be considered for the environment. There are several factors that cause these noise levels, but it is mainly in the actual moment of impact. DC generates a noise level of about 110 to 120 dB at actual source, but the duration is insignificant compared to the entire process cycle. On the other hand, the noise level generated by RIC gives a peak of 88 to 92 dB at distance of 6 m every time hammer hits the anvil. The frequency for RIC treatment is of a much higher character than for DC, which can be perceived irritating. However, the equivalent continuous level L_{Aeq} for both of the methods still meets the environmental limitations at distances of 50 m from the operation source. In order to keep a low noise level and less disturb a third party, it is essential to think of echoes, wind direction and how the base carrier is positioned (Slocombe, 2005; Adam & Paulmichl, 2007).

4.7 Practical Aspects

In addition to the above-mentioned factors, there are a lot of other practical aspects to consider when using these methods. Because large base carriers are used in the form of crawler cranes and excavators, it is extremely important to have a safe and stable working platform to stay on. If soil conditions are very wet, such a cohesive soil can be, it is generally required to introduce a granular based working carpet that is free-draining. This allows simultaneously the excess pore water pressures to dissipate more easily. A very thick backfill layer can also have a negative effect because the stress impulses may be inhibited. Therefore, it may be of a good idea to start with a slightly thinner layer of backfill and then successfully introduce new material as the work progresses (Slocombe, 2005).

A very wet weather can even causes sand-sized materials to fly far away from the compaction location through the air when pounder hits the ground. There have been times when materials have flown 60 m away from the impact point. This is only something that may happen during DC treatment. Because the impact foot of a RIC equipment is constantly in contact with the ground, the risk of flying debris does not exist. Introducing a coarser material if the work is carried out very close to inhabited areas can solve the problem. Even different types of protective screens have been used to reduce the risk of flying debris, but without any real success. They have either been too weak due to the impact velocity of the ejected particles or the constant movement of the screens has been seen as very ineffective in terms of productivity. It has been considered more effective to perform work under more dry weather conditions (Slocombe, 2005).

5 Case Histories

5.1 Case History Summaries

This section gives summaries of several case histories of projects where the soil improvement technologies DC and RIC have been used. These were selected to highlight several different cases under a certain number of different soil conditions and to give the different reasons and benefits of using these technologies as well as obstacles and drawbacks of each method. Each summary contains the following information to the extent reported:

- Location
- Site description
- Soil conditions
- Project description
- Performance requirements and design criteria
- Details on the equipment and performance
- Outcomes of the project
- Additional commitments

At the end of each case history, a conclusion between difference in the designed and achieved values is presented. It will be discussed how the values differ or, on the other hand, appear to follow each other. The designed values refer directly to the equation (4.1) and (4.4) for DC treatment and RIC treatment respectively. This will later be followed by a more general discussion in the next chapter, highlighted with tables showing the designed and achieved values as a comparison.

5.1.1 Liquefaction-potential granular backfill, Massachusetts (US)

Tan (2007) reports on a relocation project of a roadway in Massachusetts, U.S. The new road was going to be a four-lane divided highway with an intermediate area in-between. A large part of the road was constructed with embankment fills, where the major part of the underlying soil consisted of granular soils in a good quality for the foundation of the proposed road embankment. However, there were also those parts in the project that consisted of unfavourable soil found in bog and pond areas along the roadway. These areas consisted of organic peat with various thicknesses down to a depth of 10.5 m below the ground surface. The only option at these areas was to excavate and replace with a suitable granular soil due to the unpredictable settlements at long-term run. The likely future damages of differential settlements were carefully considered in comparison to the costs of mass exchange. Thus, excavation and backfill were the only option in those areas together with sheet pile walls as retaining structures, see Figure 5-1.

The newly placed backfill was thought to be of a very loose state. Because of that two passes of DC were introduced to compact backfill to mitigate liquefaction hazards. Cone penetration tests were executed both before and after the DC treatment to investigate whether the potential liquefaction soil would be sufficient densified or not with this amount of treatment. In addition, the required minimum cone resistance q_c value to resist a potential liquefaction was also presented in the paper by (Tan, 2007).

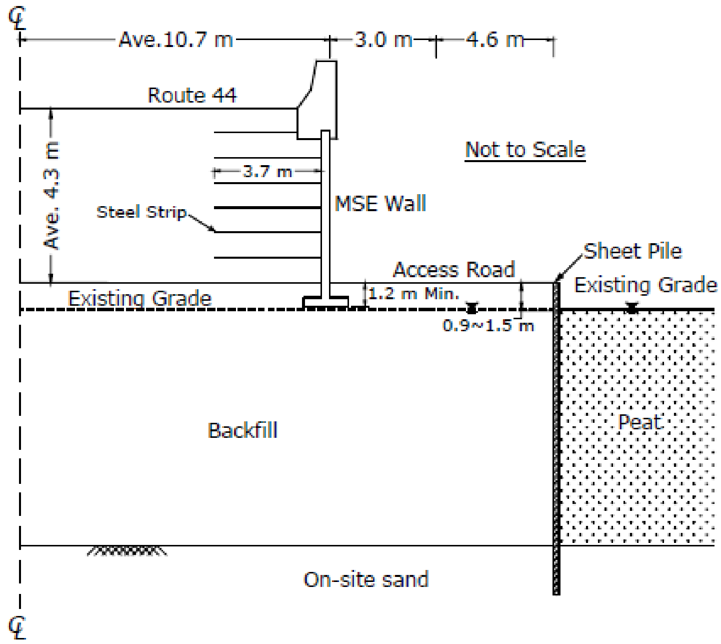


Figure 5-1: A typical road structure [10]

Before the actual compaction work took place a soil investigation of the newly place backfill was performed. This showed a backfill with a depth between 2.5 and 10.5 m consisting mainly of fine to coarse sand with some amount of gravel and silt. The backfill could later be classified as a liquefaction potential soil, which was underlain by medium to dense natural sand. Tan (2007) reports that a 14.4-ton tamper dropped from an 18.3 m height made compaction work. Two passes were used completely over the entire area with a six days' interruption between the first and the second pass. Both passes had a grid spacing of 4.6 m, the only difference were that the second pass was adjusted to a position in between the first pass. The same distance was kept from sheet pile walls as a minimum so that no damage or deformation would occur during compaction. In every tamping point nine drops were utilized.

Several cone penetration tests (CPT) were executed around the compaction area both before and after each tamping pass to facilitate the assessment of the compaction degree. Tan (2007) presented CPT results from two different stations at the site, which showed a significant improvement in the cone resistance q_c and side friction f_s of the backfill, see Figure 5-2 and Figure 5-3. As the cone penetration is a good indicator for evaluating the ground improvement, it revealed a densification in the deposit up to a depth 11 meters. No significant changes could be seen in the densification for the in-situ soil below the backfill as well as for the non-replaced peat at a depth of 9.5 to 11 m, see Figure 5-3. The compaction result indicated still a pretty uniform distribution of friction ratio R_f over the complete depth, from being kind of uneven before.

Regarding the liquefaction that was the main point of investigation in this paper. It was concluded that before any soil improvement was performed, the backfill would liquefy below the water level upon the occurrence of a designed earthquake. This would also be the situation after the first pass in most places, but after the second pass, it could be demonstrated a non-liquefiable soil down to a depth of 4 m. Under these 4 meters the likelihood of liquefaction would still be very high but an increase for the underlying in-situ soil could be seen. Finally, Tan (2007) established that a minimum cone resistance q_c value required to resist potential liquefaction hazards is around 8 to 9 MPa after using DC.

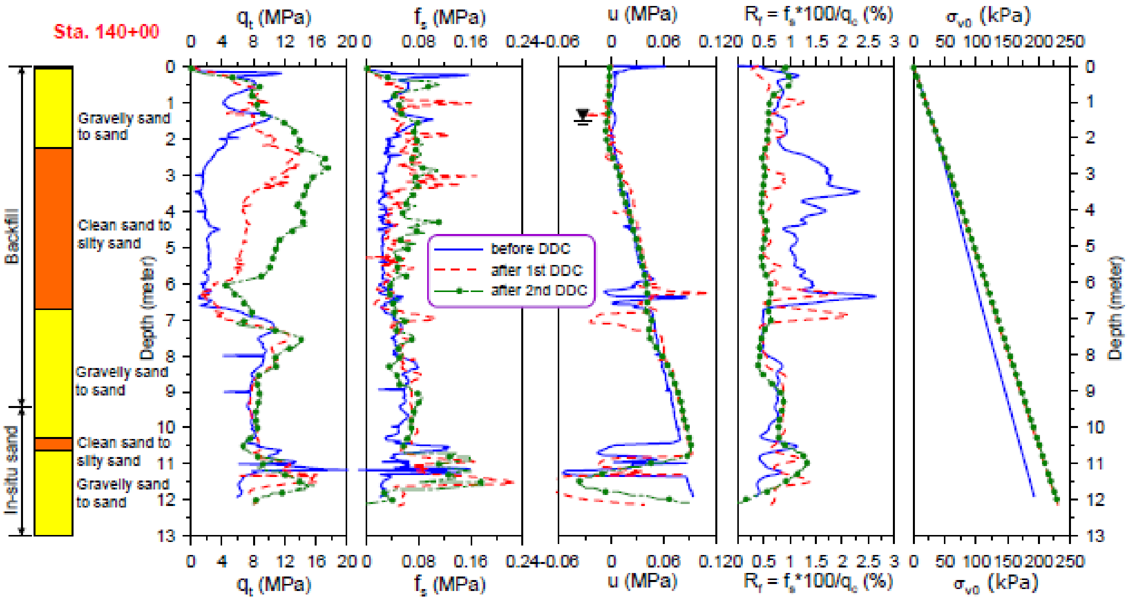


Figure 5-2: Results from station 140+00 [10]

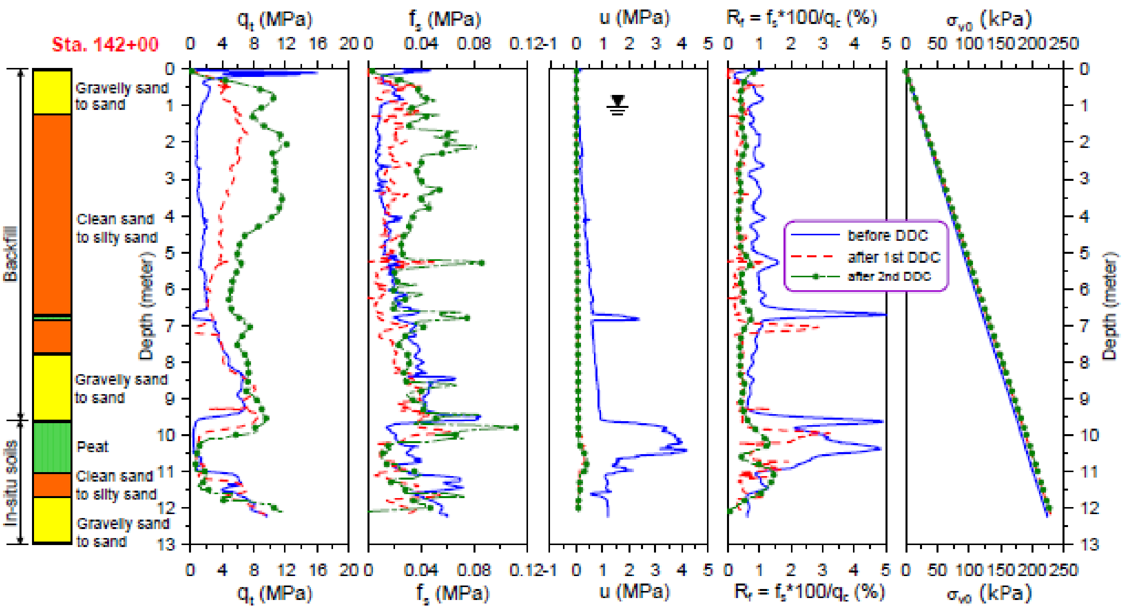


Figure 5-3: Results from station 142+00 [10]

Difference between achieved and designed values

The report presents an achieved value of soil improvement down to a maximum depth of 11 m. An own view of the result, referred to the Figure 5-2 and Figure 5-3, states clearly that the biggest change in the cone resistance values occurs at a depth of 8 to 9 meter. According to the given execution data a design value of soil improvement can be calculated. However, the empirical coefficient n_c is not mentioned in the paper, but referred to the Table 4-1 and the given description of the soil conditions a value is estimated. Thus, the designed value is as follows:

$$D_i = n_c\sqrt{W_t H_d} = 0.5\sqrt{14.4\,t\cdot 18.3\,m} = 8.1\,m$$

This indicates that both the designed and the achieved value are more or less followed. A design for DC in cases with newly placed granular backfill proves to work appropriate. The formula seems to be applicable to a quite large extent for these conditions.

5.1.2 Compaction at a quay extension project, Felixstowe (UK)

Vink & Dijkstra (2016) reported on a quay extension project at the Port of Felixstowe, which is one of United Kingdom's largest port. The project consisted of an extension of an additional quay with 190 m and 16.000 m^2. The new area (see Figure 5-4), where the actual quay was being constructed, was surrounded by tubular piles, tubular piles combined with sheet piles and sheet piles before the reclamation. The closure made it possible to fill the basin with dredged sand up to approximately +0m level according to the Chart Datum (CD). Further a stockpile material was placed on top of the dredged sand up to a level of +2m CD.

The work was performed by using the Cofra Dynamic Compaction (CDC) method, which is a heavy rapid impact compaction method that uses a 16-ton weight dropped onto an impact foot from a height of 1.2 m, see Figure 5-5. Vink & Dijkstra (2016) mention further that the process was made with the aim of reducing future settlements and improving the properties of the sand. Extensive compaction trials were performed before the actual compaction work could start. These were done to determine how well the compaction process in relation to the requirements proceeded.

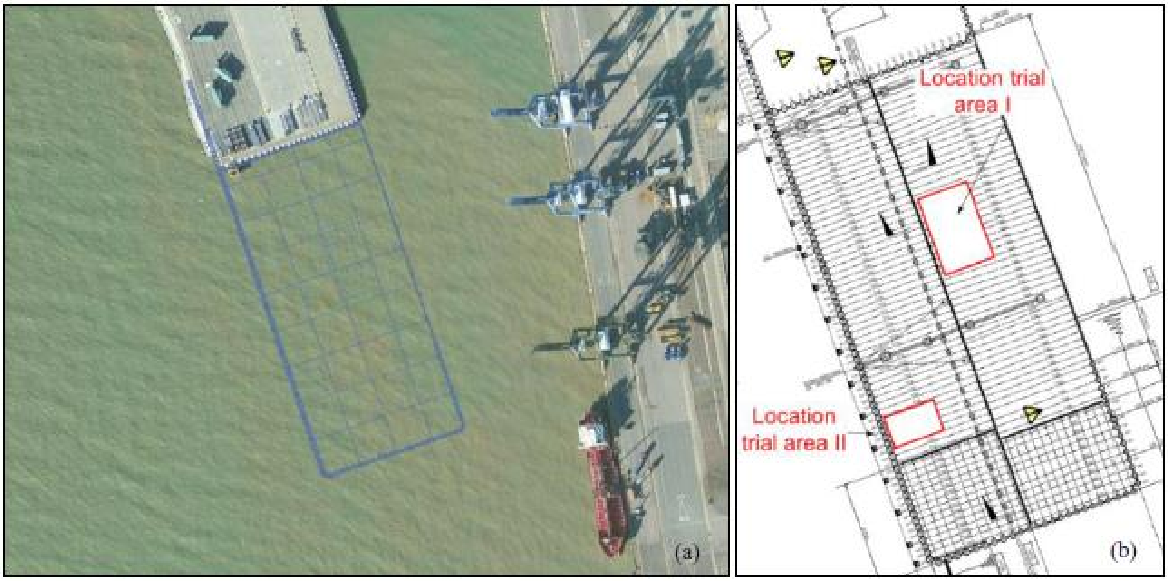


Figure 5-4: a) Overview of quay extension b) Location of trial areas [11]

The requirements that were set for the compaction of the fill at the beginning were determined from two boundary conditions. One was about the future settlements, which were limited to 7 cm over 2 years and 13.5 cm over 20 years thereto some settlements from the London clay below the structure may occur. The second was about the minimal friction angle profile of sand. Both the actual allowable settlement and the friction angle were translated to a minimal cone tip resistance (q_c value) profile and later combined, see Table 5-1. The requirements for the deformation of the wall during compaction were set to 50 mm.

Table 5-1: Design parameters for the project [3]

From [m CD]	To [m CD]	Relative Density [%]	Cone Resistance (q_c value) [MPa]	Internal Friction Angle [Degrees]
2.0	-3.0	60 – 70	10 – 15	42
-3.0	-5.0	30 – 40	5 – 6	35
-5.0	-12.5	20 – 30	4 – 5	32

During the compaction trial both the grid pattern and stop criterion was set, considering the required CPT profile. Thereto the influence on the lateral compaction was checked to investigate the shortest possible working distance from the wall. Because of the changing site conditions in form of the tidewater (see Figure 5-5), a bit less complex stop criterion must be used. The more advanced stop criterion settlement per blow could not be used since the reactions of the impact foot during compaction differed so much due to the high and changing water levels. Otherwise Vink and Dijkstra (2016) suggest that a criterion like that is very useful for receiving a homogeneous compaction, but it requires during the compaction trial some statistical analysis of post compaction tests.

Vink & Dijkstra (2016) point out that there is no relation between the three stop criteria (total settlement, settlement per blow and number of blows) in this project, which is usually used to be in cases under homogeneous conditions. Finally, the stop criterion was decided to be in the amount of blows just to insert enough energy into the ground. Around 40 to 45 blows, seen from the published logger data in paper (Vink & Dijkstra, 2016). Furthermore, the grid spacing came out to be of the size 2.75 m by 2.75 m.

Figure 5-5: The RIC equipment working at site [11]

In Figure 5-6 the cone resistances are presented from the quay extension project, where only small to no differences in q_c values between the CPT locations occur. Three measurement points were placed both at a compaction location and around a compaction location. A satisfying result shows that a homogeneous compaction has been performed due to a sufficient overlap in the influence zones between compaction locations, and additionally the compaction criteria have been reached over a good depth. There is just a small dip in the cone resistance at the level of -3m CD. In the report (Vink & Dijkstra, 2016), it is written that the dip depends on a local inclusion of finer materials.

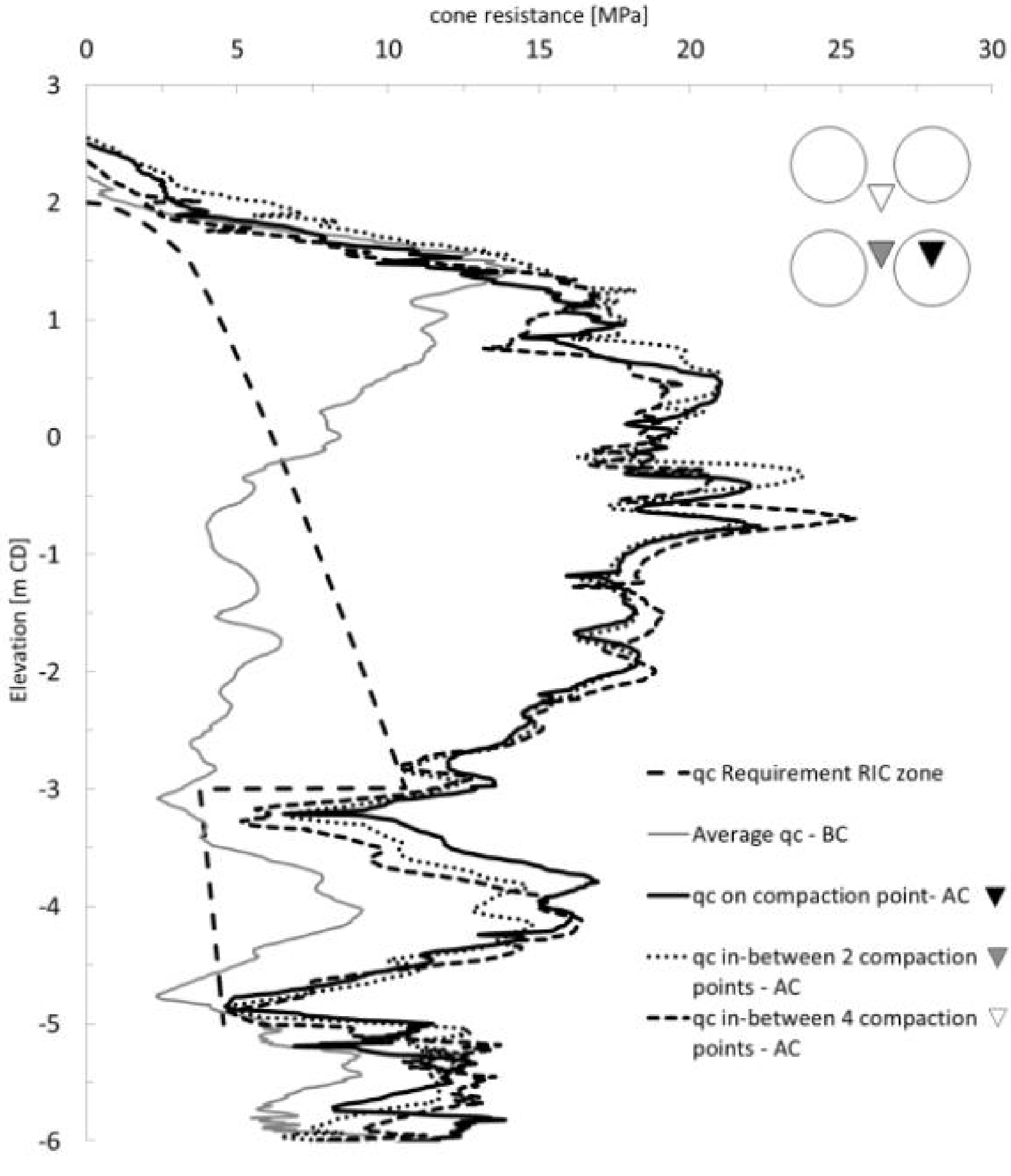
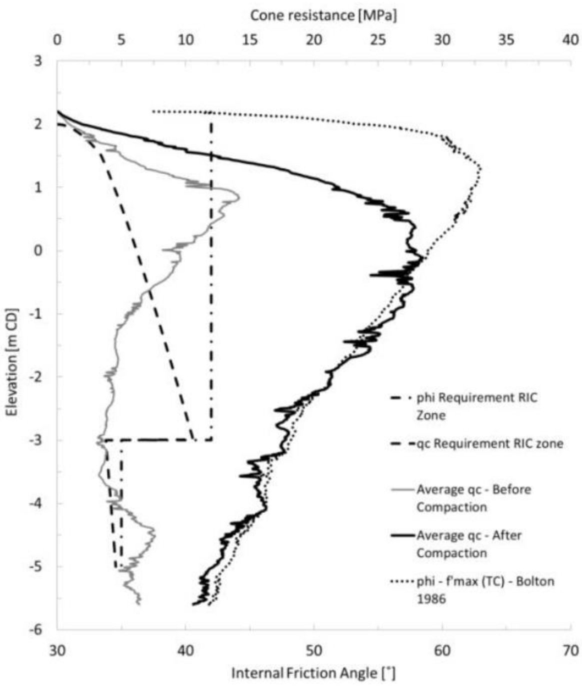


Figure 5-6: Cone resistances at the compaction location and around the compaction location [11]

Even the deformation measurements of trial compaction next to the main quay wall show an appropriate result. Several different trial setups were used just to investigate if there could have an effect on the deformations of working from or towards the wall as well as impact from the tidewater. The work was done very close to the wall, which can be seen in Figure 5-5, the closest locations were approximately 40 cm from the quay wall. The deflections of the main quay wall kept within 40 mm, which is in an acceptable limit in view of the requirements.

Finally, Vink & Dijkstra (2016) reported both the average cone resistance (q_c value) of all the 31 CPT's and the average internal friction angle taken from the field area after the actual compaction was executed, see Figure 5-7 a). They show a result that is well above the requirements. Figure 5-7 b) displays the influence of the compaction over depth in a single location. The authors mentioned that the compaction treatment reached down to depth of about 8 to 10.5 meter.

a)



b)

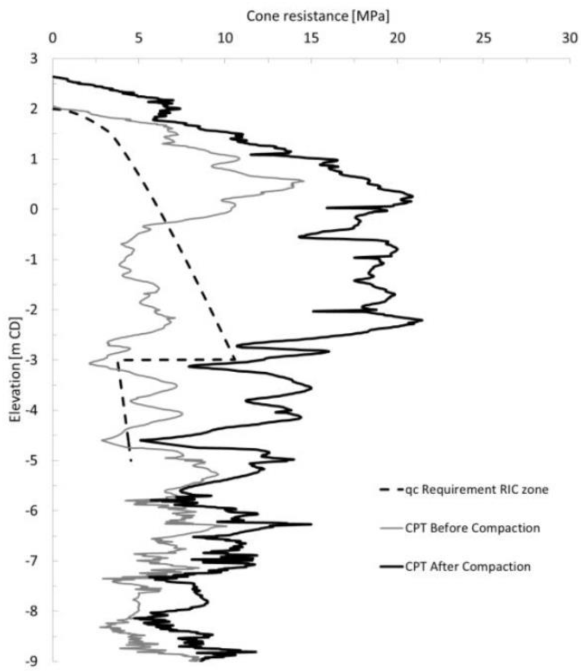


Figure 5-7: a) Average cone resistance of all compaction b) Influence of the compaction [11]

Difference between achieved and designed values

The authors mention a soil improvement result of 8 to 10.5 meters' depth; there the largest improvement occurs in the top 6 to 7 meters, see Figure 5-7 b). After calculating the design value of soil improvement out of the given data, the following results were obtained.

By calculating the impact velocity first, it is possible to calculate the resulting momentum. Table 5-2 has finally compiled for the depth of improvement at three separate increases of relative density. The equation (4.4) has been used to calculate the treatment depth, as shown below. Depth correlation coefficients are displayed in Table 4-2.

$$v = \sqrt{2gH_d} = \sqrt{2 \cdot 9.81 \frac{m}{s^2} \cdot 1.2\, m} = 4.85\, m/s$$

$$M = W_t \cdot v = 16\, t \cdot 4.85 \frac{m}{s} = 78\, ton\, m/s$$

$$D_i = a_Z + b_Z \log(N_d \cdot M)$$

Table 5-2: Designed value of soil improvement

Increase in the relative density *(%)*	Treatment depth *(m)*
40	4.2
20	5.3
10	6.3

The upper layers in the deposit have obviously greater increases in the relative density than the designed value mentions. For example, the depth around 4 m and 5 m in Figure 5-7 b) seem to have a much greater increase than 20 to 40 %. It can also be concluded that soil improvement below the designed value (below 6.3 m) occurs, but somehow it is minor. There are some sufficient increases in the cone resistance values further below these 7 meters as the writers say. This is probably related to the soil conditions. The designed value of RIC treatment does not take the soil type into account, which perhaps makes the designed value misleading for some cases. Since RIC method seems to work pretty well in cases with the dredged sand, the designed value does not really match the achieved value exactly.

5.1.3 Dynamic compaction in loose granular deposits, Arabian Peninsula

Kozompolis, Vettas & Chlimintzas (2013) write about a soil improvement project in the coastal area of the Arabian Peninsula that would provide foundation support for a whole new infrastructure. The site soil conditions near the surface consisted of aeolian sands and dune sands, which are of a poor character. Under this top layer sabkha saline clay and silt were situated and further down at greater depths dense sand and hard clays were found.

a)

b)

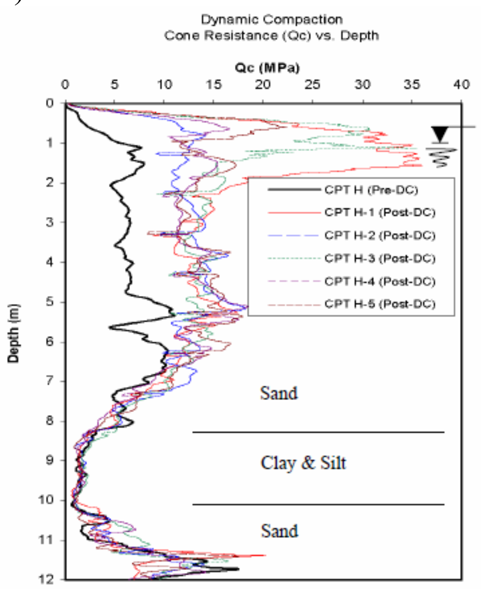
Figure 5-8: The both devices in action at the site a) Dynamic compaction b) Rapid impact compaction [12]

A detailed soil exploration tells that the site was covered by four to eight meters' thick layer of loose to medium dense, poorly-graded sand with silt. In some places, even a thin silty/clayey layer (approx. 1 m) lay on top of it. Two to three meters under the top granular layer the soil consisted of a low to medium plasticity/medium stiff to stiff layer of silt and clay. And finally, at the deepest bottom a very dense sand and hard clay layer was encountered.

The soil improvement work for this granular soil was developed in order to manage the induced loads from the shallow foundations of buildings, keep the expected total and differential settlement within an acceptable limit and reduce the potential liquefaction in a case of mild seismic situations. Dynamic compaction considered being a satisfying option in the case, and it resulted in an adoption of two different dynamic compaction methods. Kozompolis, Vettas & Chlimintzas (2013) emphasize that both DC and RIC methods came to be used in the project to conduct a cost-efficient work that would be ready in time. DC with a weight of 20 metric tons and a drop height of 23 meters was used at areas where the thickness of improvement target zone was down to about 8 meters, while in areas where the thickness varied between 4 to 6 meters RIC was applied. In the Figure 5-8 it can be seen the both devices working at the site.

Cone Penetration Tests (CPT) were executed over the entire site both before and after the soil improvement for the quality control. Based on the cone tip resistance (q_c) values of CPTs the soil improvement indexes were calculated and plotted in a graph showing the depth of improvement. In the Figure 5-9 a) DC's cone resistance values from some of the neighbouring locations can be seen with an average of 13.5 MPa. In the upper part of the deposit some soil disturbance has occurred and should thus not be taken into consideration. The soil improvement indexes for exact the same locations are published in the Figure 5-9 b), which in turn give a depth of influence around 7.5 and 8.0 m. Kozompolis, Vettas & Chlimintzas (2013) point out in the report that the average depth of influence where DC has been utilized is between 7.0 and 8.5 meters.

a)



b)

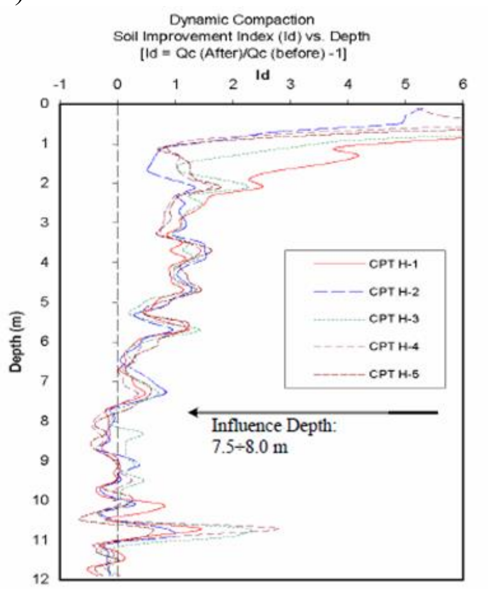


Figure 5-9: a) CPT before and after soil improvement b) Soil Improvement Index [12]

Similarly, the quality control for the RIC treatment area has been performed by using CPT tests. The treatment area was divided into the three different areas with regard to the pre-improvement in-situ soil state and the stratigraphy. Both cone resistance values and soil improvement indexes from the three areas are displayed in the Figure 5-10, Figure 5-11 and Figure 5-12, where also the depth of influence has been shown using the indexes.

Area 1 and 2 are of the same character regarding the subsoil stratigraphy, while the possessing pre-improvement in-situ density for these two differ slightly from each other. Thus, different amount of energy is needed to achieve the soil improvement target of 12 MPa. In Area 1 where the cone resistance values were slightly lower, it was found that 400 to 570 ton-m/m^2 would be required to achieve the target, while in Area 2 there it turned out that 70 to 250 ton-m/m^2 would be enough. The average post-improvement cone resistance values for the areas came out to be between 16 and 20 MPa for Area 1 and between 15 and 18 MPa for Area 2.

On the other hand, in the third and last area (Area 3), the depth of improvement was limited after the appearance of a clayey / silty layer at the depth of four meters. Further, the weak soil layer reached down to 6 meters' depth and no more improvements could be presented. The required energy to achieve the target within the area was found to be between 150 and 300 ton-m/m^2. The treatment presented a mean cone resistance values between 17 and 26 MPa in Area 3.

a) b)

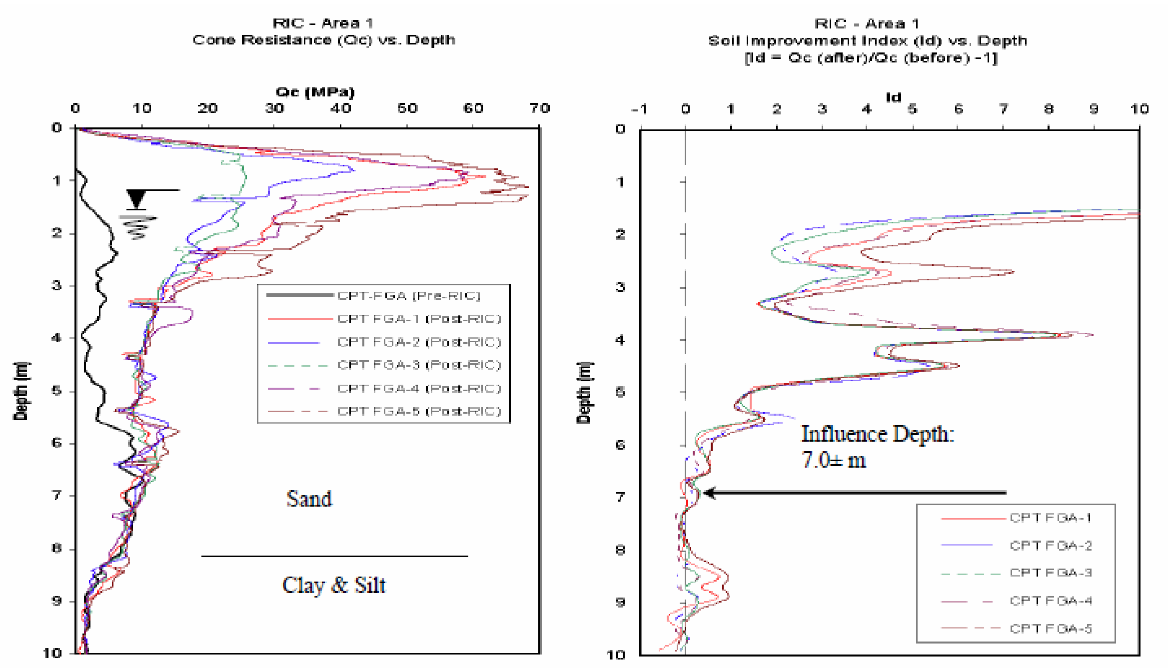


Figure 5-10: RIC – Area 1 [12]

a) b)

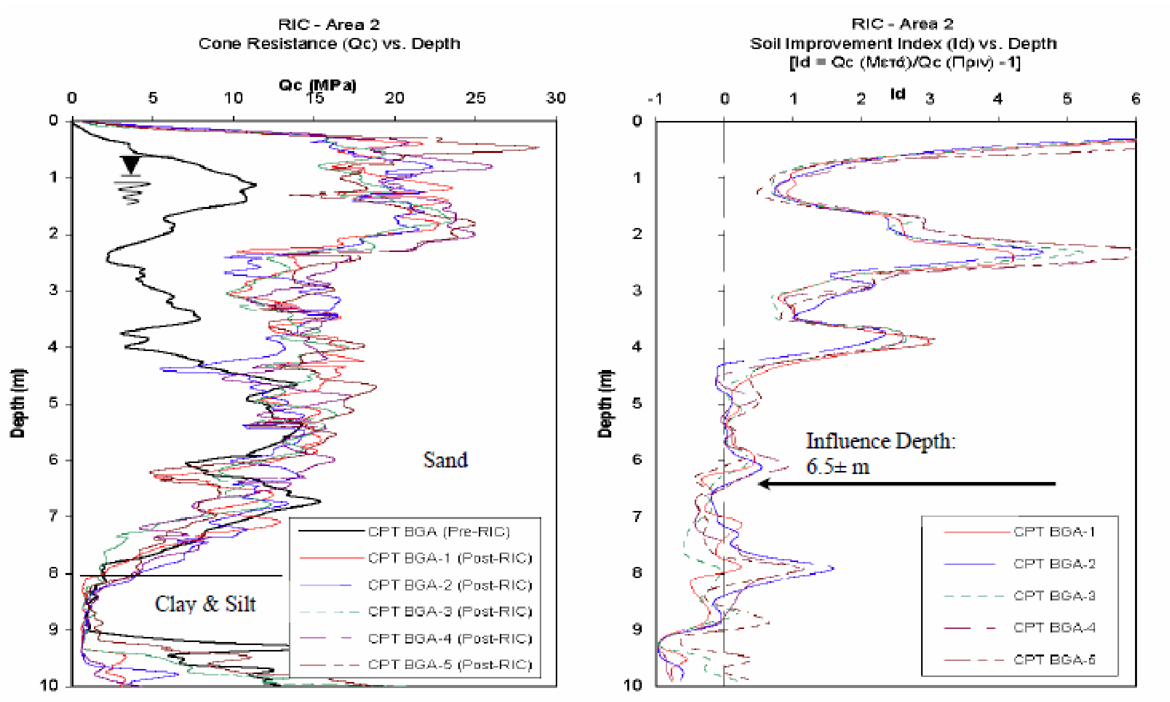


Figure 5-11: RIC – Area 2 [12]

a) b)

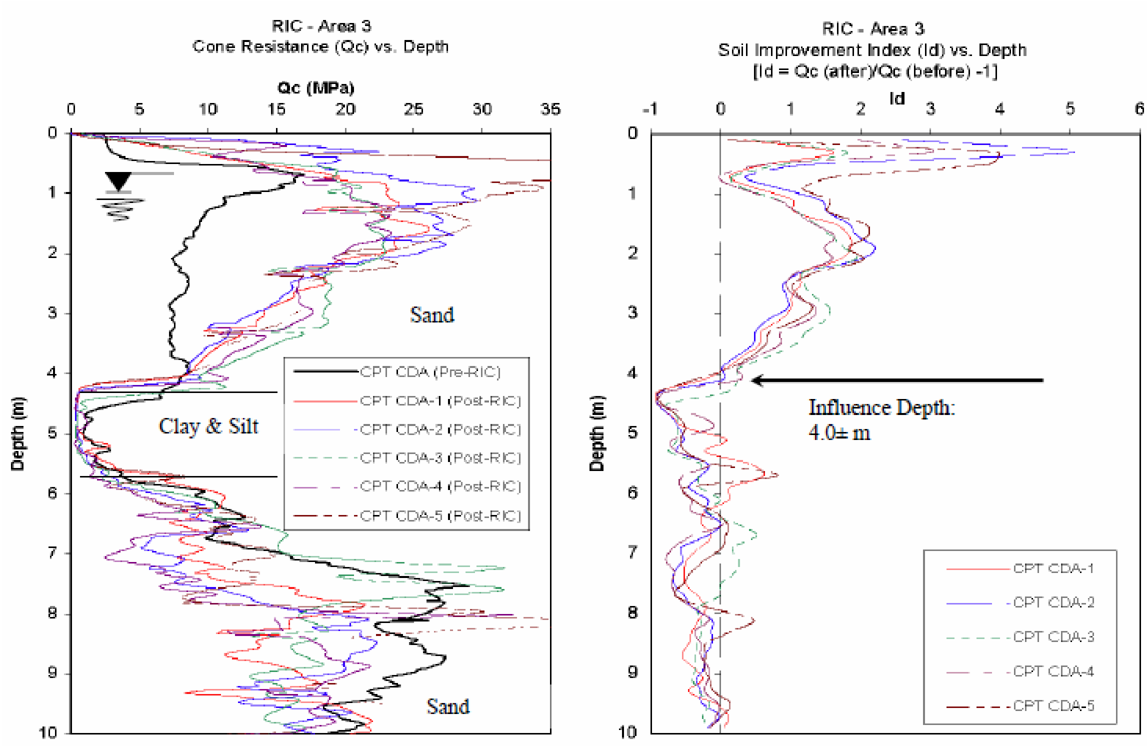


Figure 5-12: RIC – Area 3 [12]

Finally, Kozompolis, Vettas & Chlimintzas (2013) pointed out in their report that RIC method is of a much more favourable character, when it comes to the soil improvement costs. It is reduced as much as 60 percentages of the corresponding costs of DC method. But they also mean that still never forget the targeting zones as well as the availability of the device. The mobilization costs can be high.

Difference between achieved and designed values

Out of the report it is revealed that the DC treatment shows a result between 7.0 and 8.5 m. Kozompolis, Vettas & Chlimintzas (2013) establish a correlation coefficient for the soil type that is around 0.37 to 0.4 in areas where DC method is performed. This gives a designed value as follows:

$$D_i = 0.37 \; to \; 0.4 \sqrt{20.0 \; t \; \cdot 23.0 \; m} = 7.9 \; to \; 8.6 \; m$$

Thus, it ends up in the same range as the achieved value. The variation between the designed value and the achieved value is negligible in such a soil condition that is described.

The RIC treatment says, in turn, improve the soil down to a depth of 6.5 to 7 m in some area. This is perceived to be somehow overestimated, because a review of Figure 5-10 to Figure 5-12 shows results around 4 meters and not deeper than 6 meters for the achieved values. Further, the value of design is calculated. It turned out in the text that nothing was mentioned about the used equipment, only the applied energy AE for the RIC treatment. This resulted in a preparation of a cumulative number of the weight (including all blows for all passes) out of the applied energy. Thus, the used grid pattern is initially estimated. In such a way, a design value could be shown. The drop spacing is estimated to be of a square pattern 2.0 m by 2.0 m. There the hammer is dropped from a height of 1.2 m. Out of the equation (4.5) a cumulative number of the weight is demonstrated, shown in Table 5-3.

$$AE = \frac{N_d \cdot W_t \cdot H_d \cdot P}{A_e}$$

$$N_d \cdot W_t \cdot P = \frac{AE \cdot A_e}{H_d}$$

Table 5-3: Cumulative number of weight

Area for RIC treatment	Applied energy AE (ton-m/m^2)	Cumulative number of weight (ton-blows)
1	*400 – 570*	1333 – 1900
2	*70 – 250*	233 – 833
3	*150 – 300*	500 – 1000

Applying the compiled data into the equations below, it is possible to receive a value of the designed depth of improvement. The depth correlation coefficients are taken from the Table 4-2.

$$v = \sqrt{2gH_d} = \sqrt{2 \cdot 9.81 \frac{m}{s^2} \cdot 1.2\, m} = 4.85\, m/s$$

$$M = W_t \cdot v$$

$$D_i = a_Z + b_Z \log(N_d \cdot W_t \cdot v)$$

Table 5-4: Designed depth in RIC treated areas

Area for RIC treatment	Cumulative number of weight *(ton-blows)*	Increase in the relative density *(%)*	Treatment depth *(m)*
		40	5.1 – 5.6
1	*1333 – 1900*	*20*	6.3 – 6.9
		10	7.5 – 8.2
		40	2.6 – 4.4
2	*233 – 833*	*20*	3.3 – 5.5
		10	4.1 – 6.6
		40	3.7 – 4.7
3	*500 – 1000*	*20*	4.6 – 5.8
		10	5.6 – 7.0

These designed values show a result far greater than the achieved values. Thus, the designed values seem to be untrustworthy. However, the applied energy may have met a threshold energy in that sense the RIC method is not able to reach deeper layers than approx. 6.0 m for these soil conditions.

5.1.4 Ground improvement for a tank terminal, Amsterdam (The Netherlands)

A new large tank terminal for gasoline storage was going to be built at the Port of Amsterdam. This required an extensive soil improvement work, which is reported by Dijkstra and Nooy van der Kolff (2012). A risk of differential settlements underneath the storage tanks made the reason to perform the soil improvement work. A thorough soil exploration was done before the start to determine the site soil conditions. It came out that the compressible layer with a varying thickness consisted of peat on top of a clayey layer. The layer has been formed since a backfilling of a channel from the past. This can be seen in the Figure 5-13 highlighted with red colour. Above this layer a sand layer of 3 to 5 meter has been placed in the 1960s when the Port of Amsterdam was built. Further, only a decade back, a clayey layer was placed on top of the area during a construction of another new harbour.

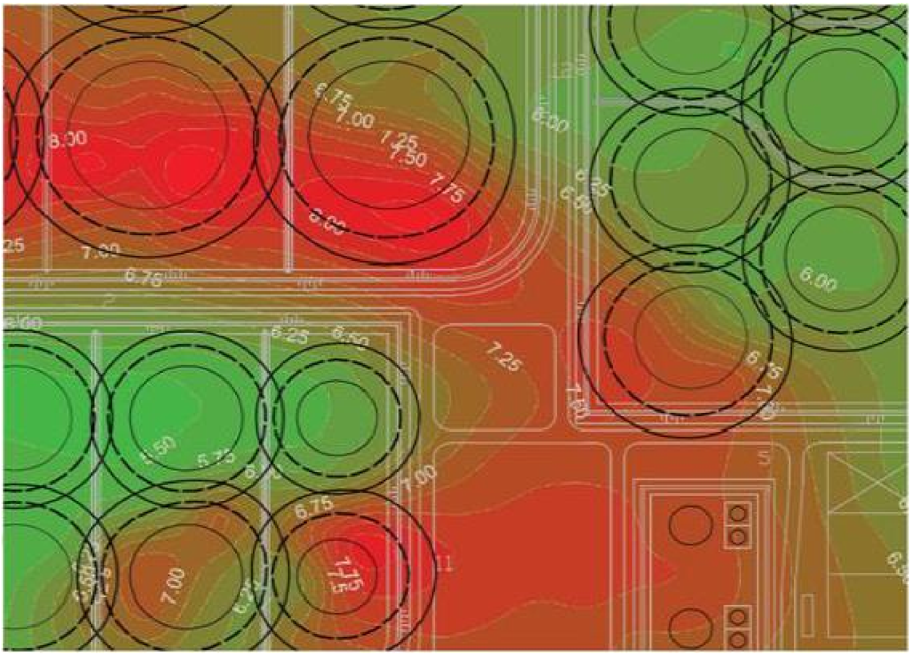


Figure 5-13: The existing compressible peat and clay layer [13]

Dijkstra and Nooy van der Kolff (2012) write that the compaction trials were performed at the site finding a solution for a ground improvement method. Both the conventional DC and the RIC method were used. The two methods were executed as a dynamic replacement technique where a coarser stone material was driven into the softer soil layer creating a stiff column to lead the load down to the bearing layers. RIC was beside that also utilized to try compacting the compressible layer. After the compaction trial progressed it turned out that only the top sand layer seemed to be densified and no material was driven into the peat layer, which can be seen in the Figure 5-15. Although the grid pattern was changed and the diameter of the impact foot was downsizing in every following compaction phase, it could not increase the capacity of the compressible layer. The improvement work caused only flooding at the surface of trial area, see Figure 5-14.

Figure 5-14: Flooding at the site due to the excess pore water pressure [13]

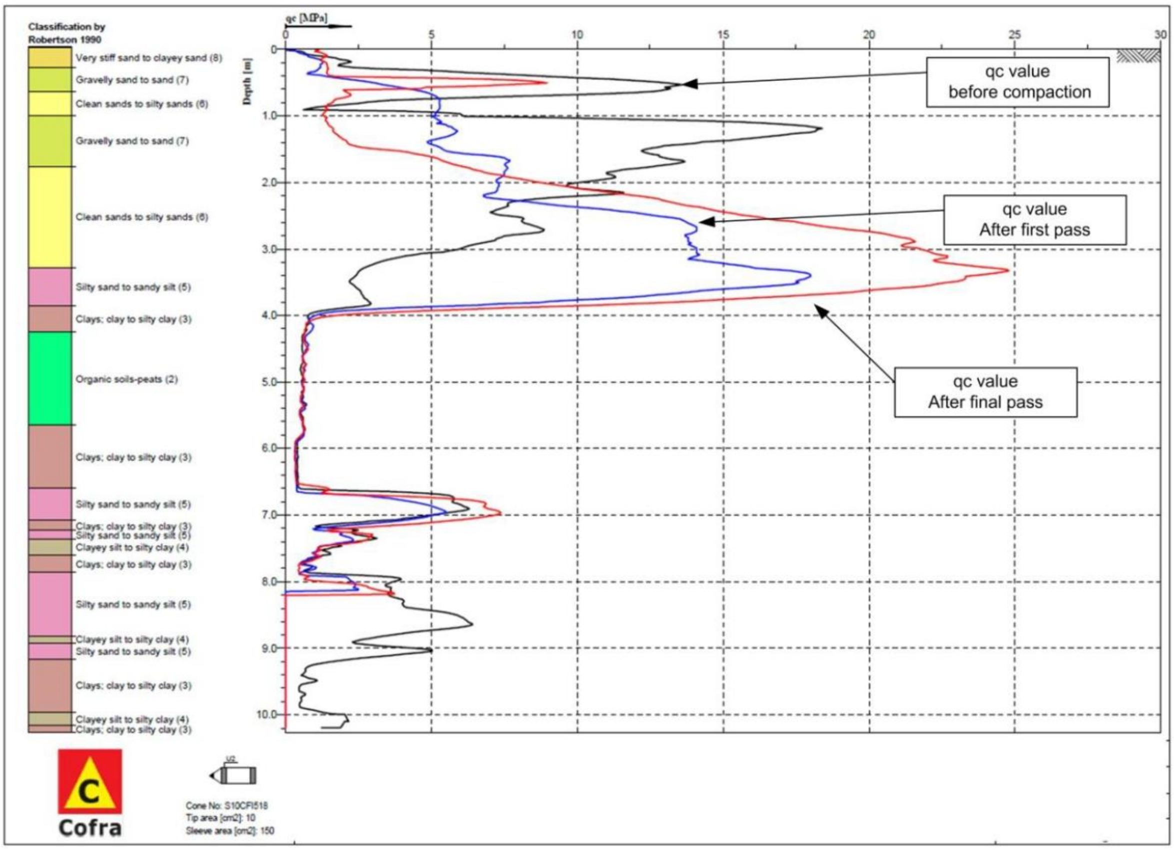


Figure 5-15: CPTs both before and after compaction trial of the compressible layer [13]

A new design was made where it was decided that all compressible layers should be removed and replaced with both local silty sand and dredged sand from nearby area, which was the reason that made the solution to be economically viable. The new solution resulted in excavation of the existing deposit from 5 up to 8 meters deep, backfilling and finally a compaction of the newly placed sand in one run. The whole process lasted for four months with a production of 2,000 m^2 per working day. No pumps were used to drain the excess water during backfilling and compaction processes, only during the excavation period to make the work more comfortable. The excavation revealed that the underlying peat and clay layer was very compact and almost dry, which was the reason why penetration of material was impossible.

The minimal cone resistance value of 5 MPa was required over the complete depth of the fill. Thereto it was decided during the trial using two kind of grid spacing on a single tank footprint to be sure a sufficient improvement was performed, one with a 3.3m centre to centre distance and another one with only 2.5m. For each grid spacing a 2-meter diameter foot was used. The grid spacing with 2.5m proved to be of great importance regarding the production and the requirements of achieving q_c values over the depth, because the 3.3m grid pattern did not meet the criteria at deeper depth, see Figure 5-16. Further, the induced settlement (23 mm per blow) was used as stopping criterion getting a more homogeneous compaction, because the varying quality of the sand and the different methods of filling made it complicated using specific number of blows. The real compaction work gave a cone resistance value ranging between 10 and 15 MPa and a maximum peak value of over 30 MPa where encounter in areas with higher initial relative density. Remarkably within the project was that the dredged sand was compacted much better compared to the local sand using the same amount of compaction energy. The difference between these two sands was that the dredged sand contained no proportion of fines while the local sand had a content above 10%.

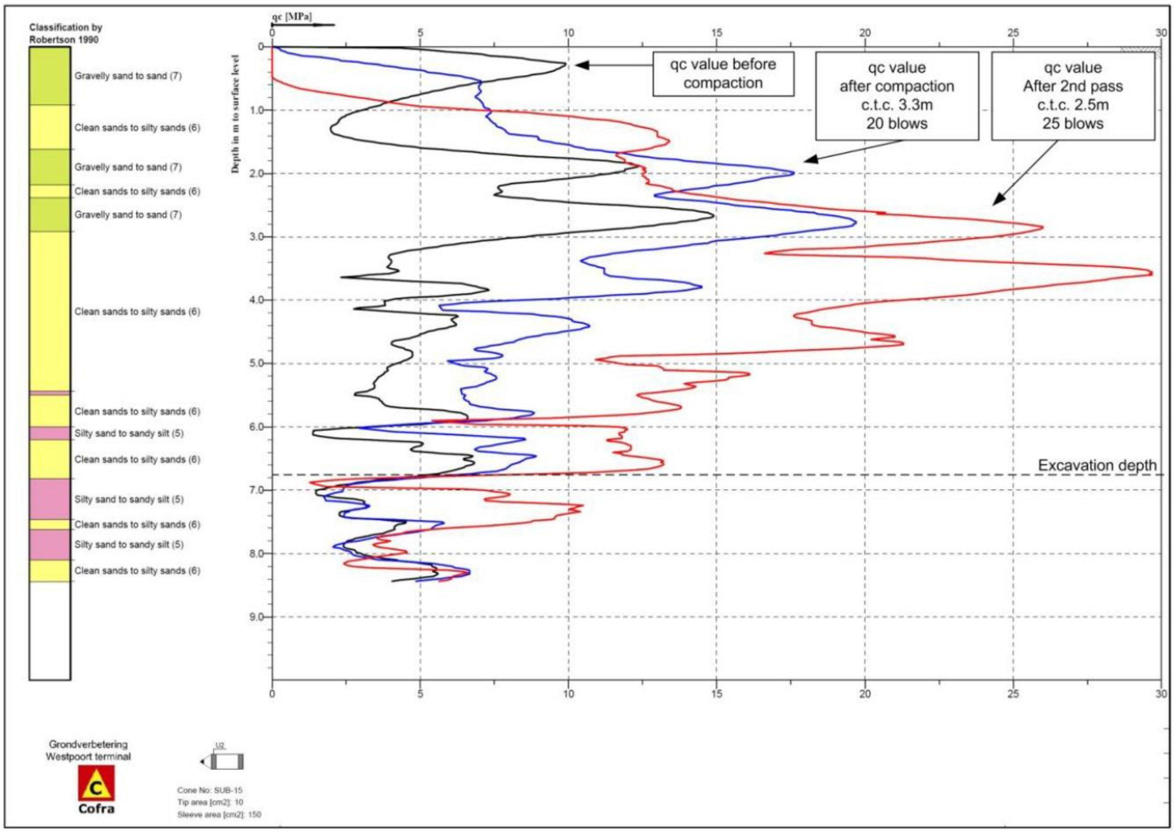


Figure 5-16: CPTs from the compaction trial determining the grid spacing [13]

Further, Dijkstra and Nooy van der Kolff (2012) mention that vibrations and displacements of four windmills critically located next to the site were monitored all the time during the operation. The distances from the compaction location to the windmills were as short as between 16 and 25m, but still the measurements remained within the tolerances.

Difference between achieved and designed values

The writers state an appropriate compaction to a depth of the entire height of backfill i.e. 5 to 8 meters. Furthermore, by using the given performance data and apply them in the equations (4.2) to (4.4) a design value can be displayed. From the logger data shown in the report an approximated number of 55 blows per compaction location is formed. The designed value came out to improve the soil at a depth shown in the Table 5-5.

$$v = \sqrt{2gH_d} = \sqrt{2 \cdot 9.81 \frac{m}{s^2} \cdot 1.2\ m} = 4.85\ m/s$$

$$M = W_t \cdot v = 16\ t \cdot 4.85 \frac{m}{s} = 78\ ton\ m/s$$

$$D_i = a_Z + b_Z \log(N_d \cdot M)$$

Table 5-5: Designed value of soil improvement

Increase in the relative density *(%)*	Treatment depth *(m)*
40	4.5
20	5.6
10	6.7

The result of the designed value stays basically in the same range as the achieved value. The favourable compaction result that is shown further down from the depth of the designed value depends probably on the suitable condition of dredged sand. The RIC method seems to manage those conditions fairly good.

5.1.5 Case study, Dubai (UAE)

Tarawneh and Matraji (2014) present a development project of a new residential area with space for 134 villas close to the city centre of Dubai. The exploration of the soil conditions at the site showed that a loose to very loose fine to medium-grained sand layer was in need of an improvement. The weak soil layer ranged from a depth of approx. 1 to 4.5 m below the surface, which also was represented by Cone Penetration Test (CPT). It can be seen in the Figure 5-17. Additionally, it was established that no cohesive soils were encountered in the area. The ground water level was situated between 0.5 and 2.0 m below the ground surface.

Design criteria was set before the project could commence with respect to the loads originated from the residential houses via foundation as well as the settlement criteria. It was later found that the soil conditions in some cases could not withstand the loads and the differential settlement exceeded the minimum limit. This led to an investigation for a suitable solution that would be both time efficient and keep costs down. Several solutions came up, but at last a thorough evaluation of three different soil improvement methods was made; vibro-compaction, dynamic compaction and rapid impact compaction. According to the evaluation, using a 9-ton RIC hammer proved to be the most suitable method when considering both technical constraints and productivity related aspects as well as costs in this project. The production rate of RIC method in this project was estimated to 2,500 m^2 per shift per machine, while only 1,000 m^2 by using DC method. Even lower when considered the method of vibro-compaction. The disadvantage of implementing RIC treatment in this project was the shallow ground water level, so it was necessary to install four well-point pipes near each villa to reduce the build-up of pore water pressure during compaction.

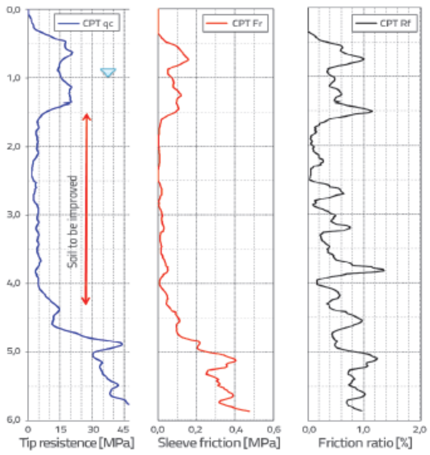


Figure 5-17: Pre-improvement CPT tests [14]

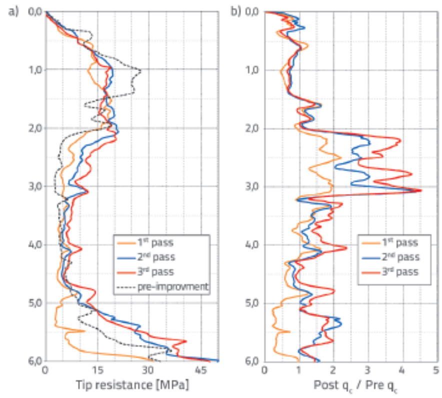


Figure 5-18: a) CPTs before and after the improvement b) Ratio of post- and pre-improvements [14]

Compaction trial was done at the site to set the performance requirements for the project. The compaction process was arranged in a square pattern using a grid spacing of 6 m at first sequence and then a smaller grid 3.0 m by 3.0 m, which came to be as one pass. It was perceived in the trial that three passes are needed using the minimum foot settlement of 8 mm as stopping criterion for meeting the design criteria of the project. The pre- and post-improvement from the trial can be viewed in the Figure 5-18. Tarawneh and Matraji (2014) point out in the report that the first pass clearly indicates a crush of the upper hard layer while following passes improving the soil. Additionally, it can be noted that a drop in cone resistance occurs at the depth of 5 to 6 meter after the first pass. The authors say that it arises since the profile of soil was pushed down after the compaction of the loose soil, but after the following passes the soil was improved again.

Since the compaction trial was done the main compaction work could be executed over the entire site area using the given compaction criteria. Ongoing monitoring and testing were all the time used during the compaction work to ensure that an adequate work was performed. The degree of compaction was assessed by comparing the cone resistance values (q_c) from pre- and post-CPTs. The post-CPTs were taken two days after the compaction was finished to let the build-up pore water pressure dissipate properly. Comparisons of some CPT results taken from three different locations at the site are shown in the Figure 5-19. It is stated in the report (Tarawneh & Matraji, 2014) that the degree of compaction reaches down to a depth of 5.0 m and with a major improvement of the loose soil between 1.5 m and 3.5 m. Notable is to mention that the biggest changes in the improvement occurred where the friction ratio of the soil was less than 1 %.

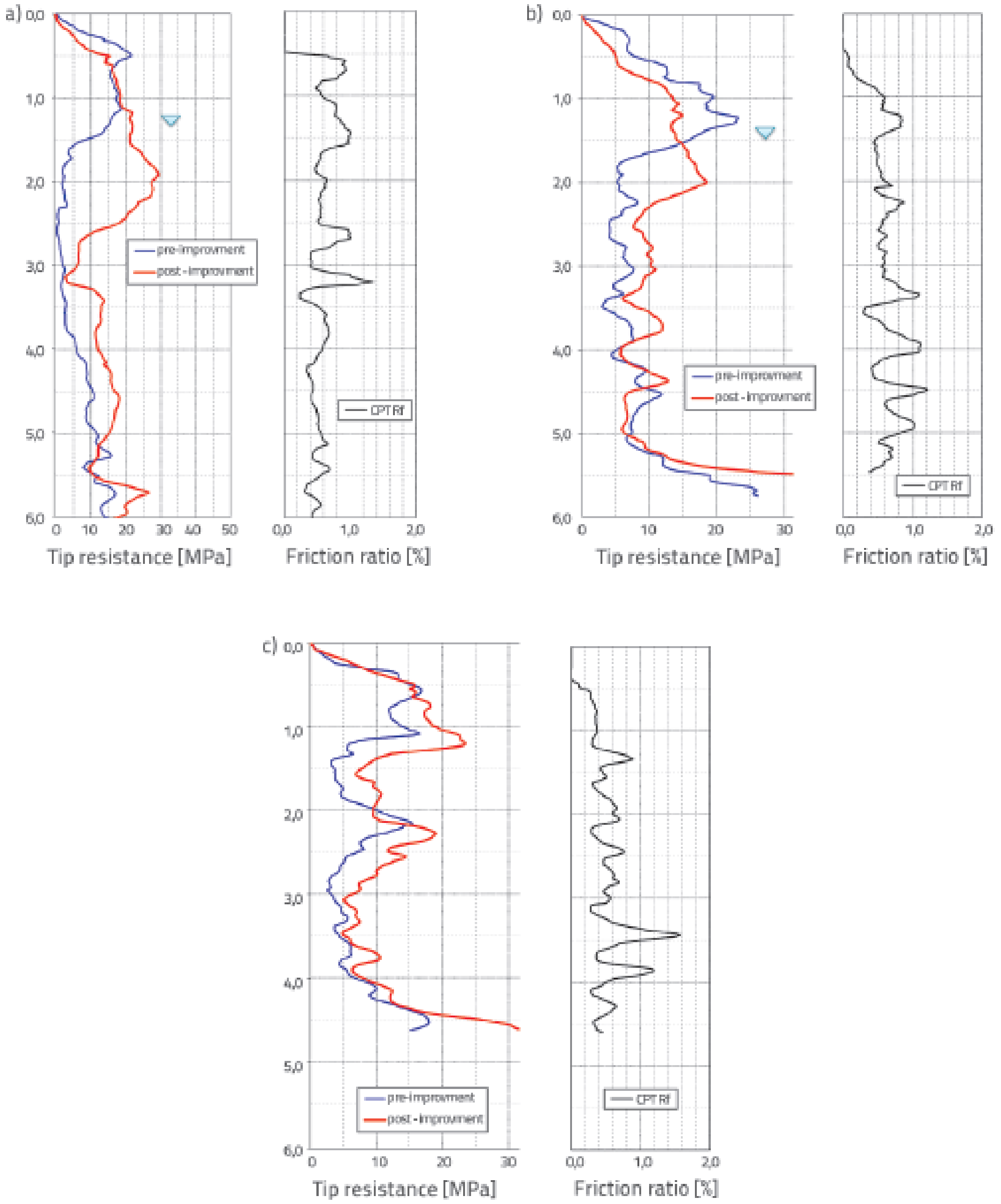


Figure 5-19: Cone resistance (q_c) values both before and after the main compaction work [14]

Vibration monitoring was made during the compaction within the trial area to detect the occurrence of possible damage to existing structure next to the project site. Seismographs were placed in a distance of 8.5 m around the compaction location to measure the peak particle velocity (PPV). The maximum PPV was measured to 20 mm/s, which was together with the frequencies far below the criteria. Thus, no constructions incurred any damages.

Difference between achieved and designed values

The achieved value indicates a soil improvement to 5 meters' depth, but it is said that the largest improvement occurs between 1.5 to 3.5 meters' depth. The designed depth is, in turn, calculated by inserting the given data into the equations (4.2) to (4.4) as well as using the correlation coefficients from Table 4-2. The authors mention there is three passes needed for the treatment, but nothing mentioned about the number of blows. A general estimation said that 20 blows per location per pass will be good. A calculation of the designed value gives a result that ends up in the quite same zone. The result can be read in Table 5-6.

$$v = \sqrt{2gH_d} = \sqrt{2 \cdot 9.81\frac{m}{s^2} \cdot 1.2\,m} = 4.85\,m/s$$

$$M = W_t \cdot v = 9\,t \cdot 4.85\frac{m}{s} = 44\,ton\,m/s$$

$$D_i = a_Z + b_Z \log(N_d \cdot M)$$

Table 5-6: Designed value using RIC in fine to medium-grained sand

Increase in the relative density *(%)*	Treatment depth *(m)*
40	3.8
20	4.8
10	5.7

This clearly presents a treatment depth that basically matches with the reported depth from the site. In some cases, it appears to possibly display too high designed values as compared to the achieved values. However, RIC method works relatively well for fine to medium-grained sand. A cooperation with well-point pipes that lowering the ground water table seems also to be well appreciated.

5.1.6 Ground improvement for a combined fire hall and office building complex, Chilliwack (British Columbia, Canada)

A case history written by Kristiansen and Davies (2004) dealing with a ground improvement work using RIC method in Chilliwack located quite close to Vancouver in Western Canada. The report is about a ground improvement for a combined fire hall and office building complex in an area of approx. 3,200 m^2. The site soil around the area indicated liquefaction susceptibility, which is something that needed to be densified. A site investigation program was adopted for the project to more properly find out the soil conditions. In several stages, it was executed via the use of diverse equipment considering the suitability of program.

The site investigation tests presented a soil condition mostly consisted of cohesionless soils. At top of the area a granular fill was situated with the thickness of 0.3 m, at one point also a soft silt could be found to a depth of 1.5 m. The surface layer was then underlain by interbedded sand and silt layers consisting mainly of sand, which reached down to approx. 3 meters' depth. It could also appear some thin cohesive silt zones in the upper part of the layer directly below the granular fill. Then from the depth of 3 meter and further down a varying graded sand layer, with minor silt and occasionally cobbles, was encountered. The sand layer occurred to be very dense down to a depth of 6.5 meters, while below that the layer was in a looser state. Afterwards from 10 meters' depth down, the granular soil was of a dense to very dense character. The groundwater was situated at a level of 2.5 to 3.0 m under the ground surface.

Soil improvement target for the project was to densify the potentially liquefaction susceptible zones located under the foundations so they agreed with the performance requirements. The building was designed to withstand a 1 in 475 year earthquake, there only limited structural damages may appear. Thus, also the ground needs to stay within the criteria. The total settlements due to the potential earthquakes should stay in an order of 50 mm to 70 mm with differential settlements of 50 mm over 10 m. As in many other projects Kristiansen and Davies (2004) write that several evaluations of different ground improvement methods were done to come up with a sufficient soil improvement method. Finally, a 7.5-ton RIC device was put into operation at the site considering the cost and vibration sensitive structures.

A pilot testing program with five compaction trial areas of 6 m by 6 m were placed around the site to set the specification for the coming RIC work. Becker Penetration Testing (BPT) was used both before and a few days later the soil improvement was performed to assess the density increase in the soil. The Figure 5-20 shares the results of the BPT tests taken from the site in five different locations. Each area had a varying number of compaction locations and passes. These are indicated in the Figure 5-20 as well as an indication of the total average settlement over the complete trial area measured after releveled the ground. The quality control data showed that the average blows per compaction location was about 30, the average final set of about 10 mm/blows due to the high silt content in many places and the average total settlement of about 0.4 m.

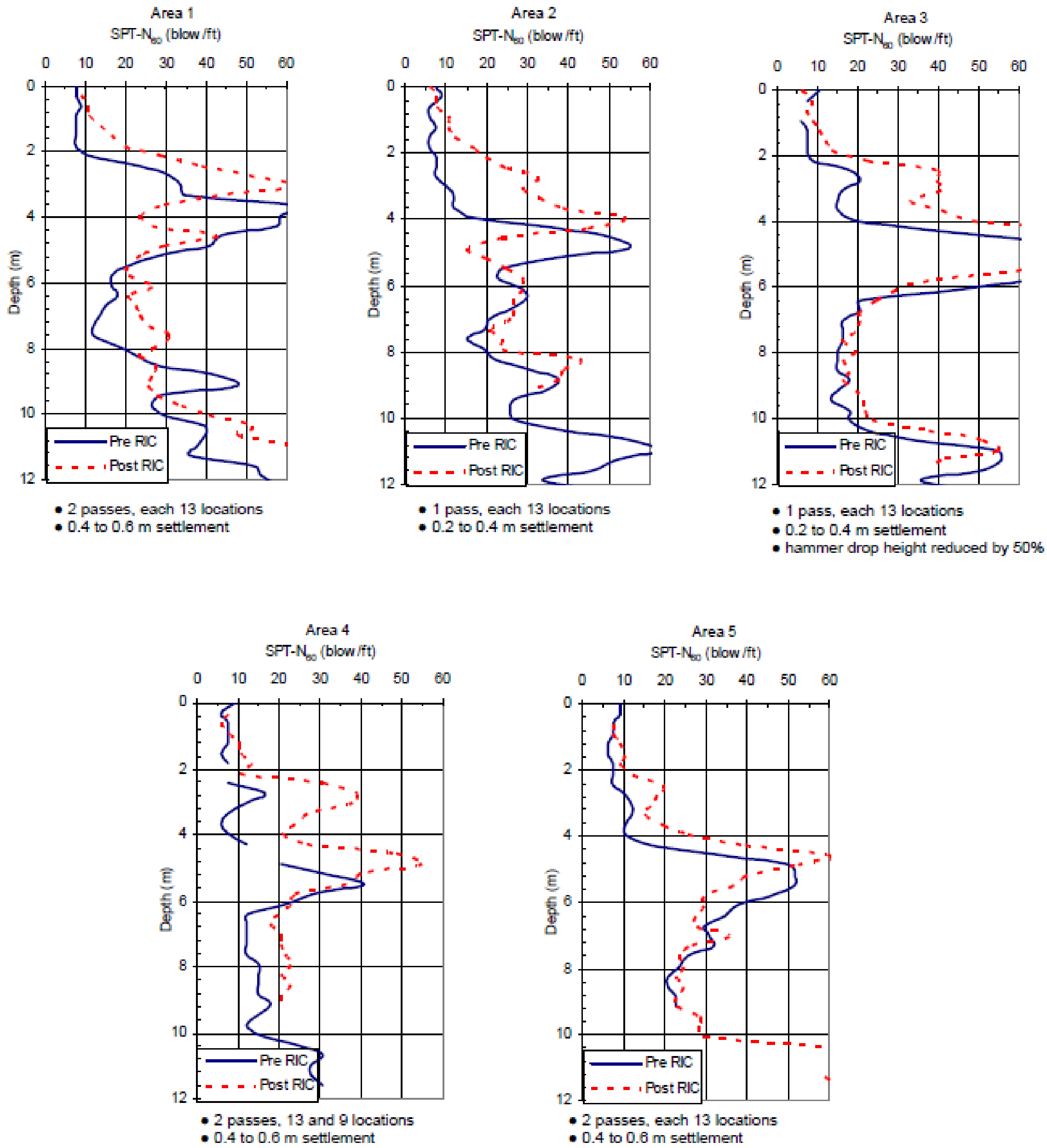


Figure 5-20: BPTs before and after the compaction trials in different locations [15]

Following RIC pilot testing program concluded that a sufficient compaction for densifying the liquefaction susceptible soil was carried out. It was established that the RIC treatments from Area 1 came to be adapted throughout the entire project area. Kristiansen and Davies (2004) claim in their paper that the post-improvement soil conditions in Area 1 shows a result there the risk of seismic liquefaction will be kept within an acceptable level at the occurrence of a 1 in 475 year earthquake, something that was the design criteria for the performance.

The wet weather conditions during the main RIC compaction work caused that the top layer containing a large proportion of fines had to be excavated and replaced with a sandy soil with minor gravel content. This was made to a depth of about 0.5 to 1.0 m, but the quality control data still showed a sufficient result in accordance with the criteria. The report (Kristiansen & Davies, 2004) also reveals that both vibration-sensitive buildings and buried utility lines were situated within a distance of 6 m and 5 m respectively. Vibration monitoring of the RIC work indicated PPV values under limitation. Opening cuts were made to reduce the impact of the compaction work.

In the Figure 5-21 BPTs before and after the RIC treatment are shown. The authors point out that there may occur some disturbance in the soil due to the content of fines in the granular deposit (soil heterogeneity). However, the depth of improvement was still great which can be seen in Figure 5-21. For the case (a) where actually the greatest influence happened is about 8.5 m, while for the other two cases (b) and (c) are 5.5 m and 6.0 m, presented by Kristiansen and Davies (2004).

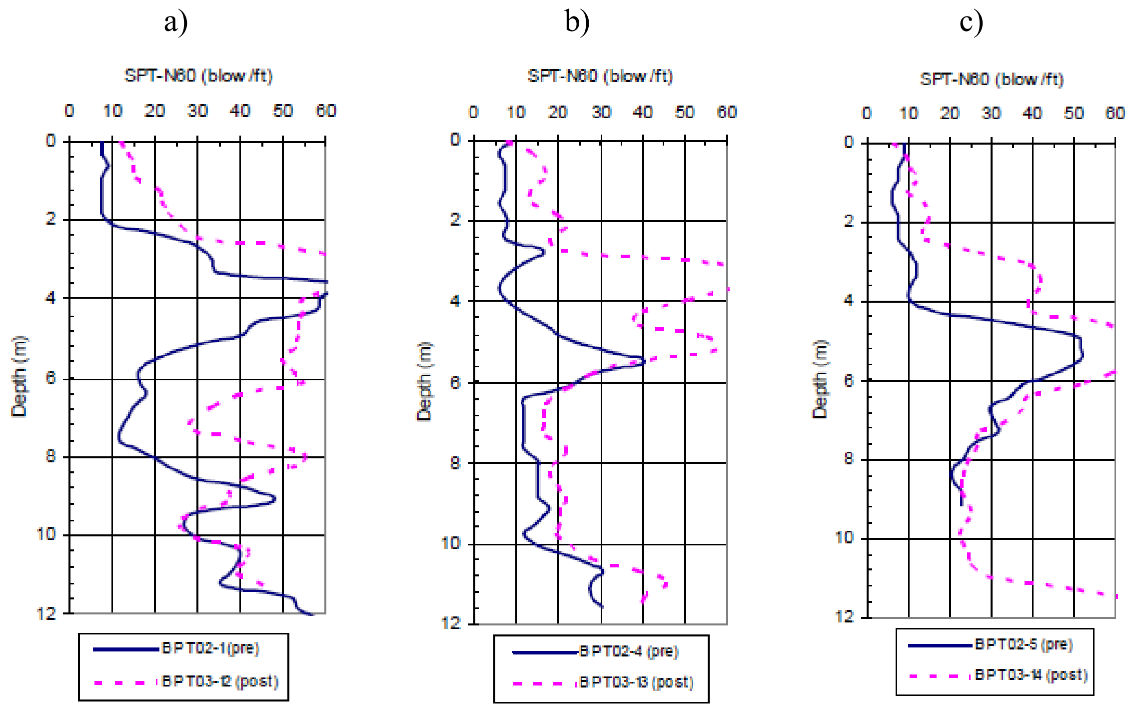


Figure 5-21: Three selected locations from the main compaction work showing pre- and post-BPTs [15]

Difference between achieved and designed values

Kristiansen and Davies (2004) show up a soil improvement work that densifies the soil to a depth between 5.5 to 8.5 m. The reported data is applied into the equations (4.2) to (4.4) to provide a design value for the RIC treatment. The designed value can be seen in the Table 5-7.

$$v = \sqrt{2gH_d} = \sqrt{2 \cdot 9.81 \frac{m}{s^2} \cdot 1.2\, m} = 4.85\, m/s$$

$$M = W_t \cdot v = 7.5\, t \cdot 4.85 \frac{m}{s} = 36\, ton\, m/s$$

$$D_i = a_z + b_z \log(N_d \cdot M)$$

Table 5-7: Designed value using RIC in a mixed soil

Increase in the relative density *(%)*	Treatment depth *(m)*
40	3.5
20	4.5
10	5.4

It shows a result of the designed value occurring more in the upper layer of the deposit compared to the achieved value. Results, like this, might make difficulties for the designed value in RIC treatment to be applied. Since the achieved value and the designed value deviate so much from each other, the reliability begins to cease.

5.1.7 A comparison of DDC and RIC methods based on field trials, Jätkäsaari (Helsinki, Finland)

Jorma Havukainen (2014) reported on a compaction trial from the district Jätkäsaari located in Helsinki, Finland. Jätkäsaari is an expansion of a former island, and land reclamation has frequently taken place during the past. Thus, the field trial was divided in five different areas to represent several types of reclaimed deposits where both DC and RIC methods were used. This made it easier to present a comparison of these two techniques considering the applicability at certain conditions.

The Figure 5-22 shows the locations of each trial site and further in the Figure 5-23 a more close-up of the specific compaction area is displayed. This shows that in every compaction area two kind of deep compaction methods (DC and RIC) were implemented and an empty space had been placed in between the two compaction tests to prevent them from being affected by each other.

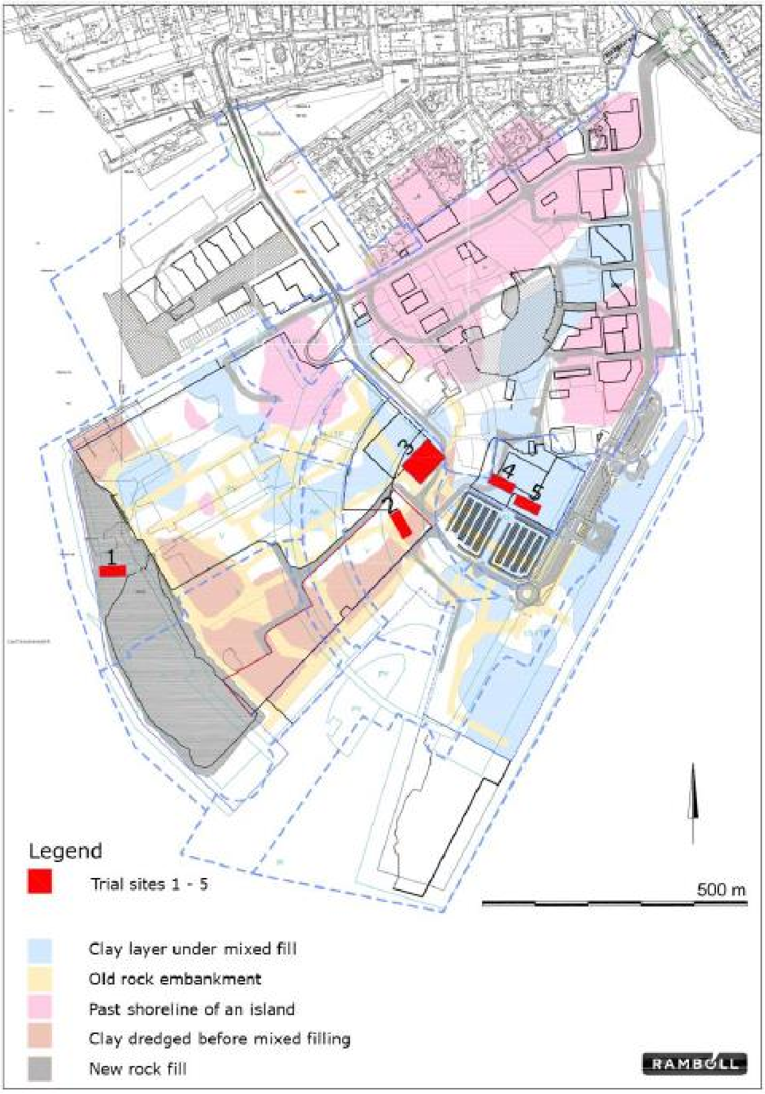


Figure 5-22: Soil map of the site showing the location of each trial site [16]

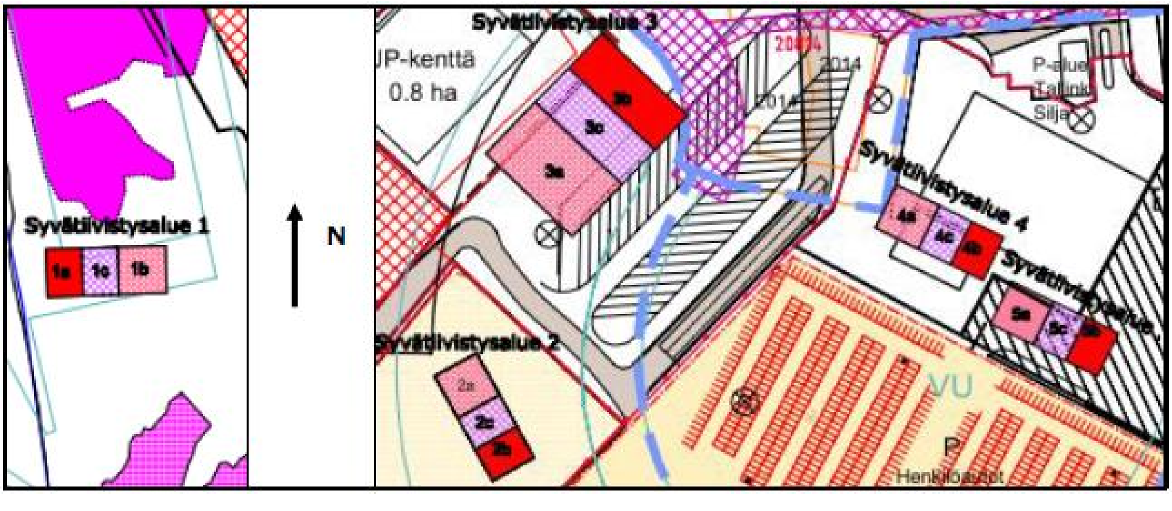


Figure 5-23: A close-up of each trial site. DC = red, RIC = pink (light red) and Non-compaction = purple [16]

An exploration of each compaction area was performed presenting that the trial site 1 completed in 2011 consisted of a rock fill with an average thickness of about 20 m. Below this filling a 10 meters' clayey layer was encountered. The clayey layer has previously been dredged from the sea bottom, while the rock fill material with stone up to 300 mm has been brought from nearby tunnel projects.

The trial sites 2 and 3 are slightly older than the first site, all the way back to 70's and 60's respectively. The trial site 2 was of a non-cohesive material with the thickness of about 10 m underlain by a 4 to 5 meters' thick layer of clay. The trial site 3 was, in turn, a mixture of sand, gravel and boulders as well as some construction waste included. The thickness of that fill layer was shifting a bit between respective ends. In the northern part the thickness was about 13 meters, while in the southern part about 19 meters. The whole trial site consisted of an underlying clayey layer with varying thickness of 4 meters in the north up to 10 meters in the south. The clay layers from both site 2 and 3 were coming from the sea bottom.

Finally, the two smaller sites 4 and 5 located close to each other have lasted there since 1930's. The both areas had a type of superstructure at top, which means that the areas consisted of a road structure. They have been in use of traffic in the past. In the trial site 4, the superstructure had a thickness of about 1 to 2 m underlain by an 8-meter layer of mixture of soil materials including wood, brick, glass, and both organic and non-organic materials. Thereto a non-cohesion soil layer was encountered deeper down as well as a clay layer. Then in the trial site 5, the superstructure was on the other hand mixed with a non-cohesive fill down to a depth of 4 to 5 m under the surface. A same kind of waste fill as in site 4 was found under the superstructure with a 10 to 12 m thickness. Under this waste fill a non-cohesive soil layer and a clay layer were also encountered.

Concerning the compaction equipment, in the areas 2 to 5, a 17-ton pounder weight was dropped from a height of 12 meter using a grid pattern 2.5 m by 2.5 m. While for the trial site 1 the same pounder weight was dropped from a 9 meters' height in the same arranged grid pattern. Beside that two kinds of complementary compaction were executed in the site 1. One high energy compaction phase and one ironing phase densifying the loose surface layer arisen from the high-energy compaction in the first phase. The complementary compaction 1 utilized a 25-ton weight from a height of 22 m in a square grid of 5 m spacing, while during the complementary compaction 2 a 17-ton weight was dropped from 10 meters' height in a half as large grid. This was executed to investigate what effect the height and pounder weight would have on the final result. The stopping criterion was set as the induced settlement (combined settlement of the latest two drops) of 15 cm or less for all trial sites.

Regarding the RIC treatment a 16-ton hydraulic piling hammer was used with a varying size of impact foot. The weight was usually dropped from 1.2 meter onto the foot resting in contact with the ground. Diverse grid patterns were used in the different trial sites, but for the impact foot with the diameter of 1.5 m, a grid 2.0 m by 2.0 m was used. On the other hand, when using 2.0 m impact foot a square grid of 2.5 m or 3.0 m was utilized. The preliminary stopping criterion for this treatment was set as number of blows (40 blows/point), but one part of trial site 3 was reserved for a special test. In that case, the stopping criterion was set as the induced settlement of 10 mm/blows or less to create a homogeneous compaction.

Havukainen (2014) writes that before the actual compaction trial could take place some asphalt removal was made where it was needed as well as the surface was graded and levelled. Soundings were also performed both before and after the compaction at all trial sites except the trial site 1 because of the rock fill. However, inclinometers were on the other hand only installed in the trial sites 1 and 2 to observe lateral displacements in the deposits, which at the same time demonstrating the depth of influence. The soundings consisted of static-dynamic penetration tests was indicating the depth of influence for the current location in sites 2 to 5. Furthermore, the settlement instruments were placed in strategic places at each site measuring the vertical displacements inside the fill.

Most of the trial areas for RIC treatment were divided into two different parts, in some cases even three. The reason was to investigate the effect of using different diameter of impact foot as well as the grid spacing. As mentioned previously the stop criterion was set to 40 blows per compaction point. After attaining the amount of blows the ground surface was graded and levelled before next pass could begin. However, in the trial sites 4 and 5 the second pass was not performed for the sparser grid pattern because of an already sufficient number of blows in the first pass and no significance in the overrun of the induced settlement. In the Table 5-8 a summary of the RIC compaction has been made showing the average settlement results from each site. The result may be a bit misleading with the respect to the entire area because the maximum settlement can be very high in some places. Especially in the site 3 consisted of very loose soil and in sites 1, 4 and 5 when examining areas with different foot diameters as well as grid spacing.

Table 5-8: Average total settlements for RIC treatment [4]

Rapid Impact Compaction • 16-ton hydraulic impact hammer • 1.2 m stroke height • 20 to 60 blows per compaction point						
Trial site	**Diameter of impact foot (m)**	**Grid spacing (m)**	**Number of blows per square meter**			**Average total settlement (cm)**
			First pass	**Second pass**	**Total**	
$1b_1$	1.5	2.0	8.7	5.4	14.1	100
$1b_2$	2.0	2.5	6.7	4.2	10.9	70
Mean values			**7.7**	**4.8**	**12.5**	**80**
$2a_1$	1.5	2.0	11.0	6.9	17.9	**71**
$2a_2$	2.0	2.5	7.7	4.8	12.5	
$2a_3$	2.0	3.0	5.3	3.3	8.6	
Mean values			**8.0**	**5.0**	**13.0**	
$3a_1$	1.5	2.0	6.3	8.1	14.4	**58** [1)]
$3a_2$	2.0	2.5	6.0	4.4	10.4	
$3a_3$	1.5	2.0	11.5	9.7	21.2	
Mean values			**7.9**	**7.4**	**15.3**	
$4a_1$	1.5	2.0	8.7	6.5	15.2	95
$4a_2$	2.0	2.5	6.7	-	6.7	50
Mean values			**7.7**	**3.3**	**11.0**	**73**
$5a_1$	1.5	2.0	10.7	8.0	18.7	65
$5a_2$	2.0	3.0	5.1	-	5.1	30
Mean values			**7.7**	**4.0**	**11.7**	**42**

[1)] Maximum settlement about 1.5 m

On the other hand, for DC treatment each drop pass was implemented over the entire area. There the pounder in the first and the third pass was dropped in the centre of grid pattern, while in the second and the fourth the pounder was dropped in the corner of grid pattern (in between the previous pass). The ground surface was graded and levelled after the third pass, and in cases where more passes were performed both gradation and levelling were done after each pass. In Table 5-9 a summary of DC compaction has been compiled. The red bold lines highlight the drop pass where the stopping criterion was met.

Table 5-9: Average total settlements for DC treatment [4]

Dynamic Compaction

- 17-ton free falling pounder
- 9 m drop height (site 1a)
- 12 m drop height (sites 2b to 5b)
- Grid spacing 2.5 m by 2.5 m
- 3 drops/point/pass

Trial site	Drop pass	Number of drops per square meter	Average total settlement (cm)
1a	**4th**	**1.56**	**55**
	8th	+0.48 [1)]	+33 [1)]
	10th	+0.78 [2)]	+17 [2)]
2b	**3rd**	**1.61**	**50**
	4th	1.76	57
3b	**4th**	**1.62**	**63** [3)]
4b	3rd	1.15	72
	4th	1.44	82
	5th	**1.88**	**93**
5b	3rd	1.19	60
	4th	**1.51**	**69**
	5th	1.95	79

1) Complementary compaction I

2) Complementary compaction II

3) Maximum settlement about 0.9 m

The depth of influence in this compaction trial has been carried out by using inclinometer measurements in the edge of test areas 1 and 2, see *Appendix 3*, as well static-dynamic penetration tests in the test areas 2 to 5, see *Appendix 4*. Both pre- and post-improvement soundings were executed nearly in the same spot to receive as good results as possible. The results from both inclinometer measurements and soundings are presented in the Table 5-10 and Table 5-11.

Table 5-10: The depth of compaction estimated by inclinometer measurements

Rapid Impact Compaction		**Dynamic Compaction**	
Trial site	Depth of influence (m)	Trial site	Depth of influence (m)
1b	9.4	1a	12.5
2a	7.9	2b	9.4

Table 5-11: The depth of compaction estimated by static-dynamic penetration tests [4]

Rapid Impact Compaction					**Dynamic Compaction**				
Trial site	**Point 1**	**Depth of influence**	**Point 2**	**Depth of influence**	**Trial site**	**Point 1**	**Depth of influence**	**Point 2**	**Depth of influence**
2a	416/424	7.9	415/425	-	**2b**	413/427	9.4		
					2c	414/426	7		
3a	423B/422	10	424/423	9	**3b**	506/420	10	69/421	8.5
4a	405/429	11	406/428	7.5	**4b**	408/431	8	409/432	8.5 [1)]
4c	407/430	8							
5a	410/433	10.5			**5b**	412/435	10.4	319/434	10

[1)] Point 409/432 is positioned between the trial sites 4b and 5a

Havukainen (2014) mentions further in his paper that the RIC compaction consisted of a 3,100 m^2 trial area that was executed within 6 days. This resulted in an average performance per day just over 500 m^2 for the RIC method. The DC compaction was performed within 11 days including the complementary compactions. The work was made in a trial area of 2,000 m^2, which led to a performance of around 200 to 300 m^2 per day. He says that the availability is crucial in terms of costs due to the exceptionally high mobilization costs.

A vibration monitoring was also made during the trial compaction to observe if any damage would have occurred in nearby buildings. However, with structures more than 70 m and even up to several hundreds of meters from the actual compaction point no damages occurred, not even small ones. Both the peak particle velocity and the frequency of vibration for the two techniques stayed in a good range to not cause any damage. Worth pointing out is that the strongest peak particle velocity for RIC occurred at site 1 (rock fill) both when using the larger as well as the smaller diameter of the impact foot (Havukainen, 2014).

Difference between achieved and designed values

The compaction results (achieved values) from the site can be studied in Table 5-10 and Table 5-11 as well as Appendix 3 and 4. Havukainen (2014) states in the paper that the achieved values from DC treatment range between around 8 to 12.5 meters. Simultaneously, it turns out that the achieved values from RIC treatment are between 7.5 to 11 meters. On the other hand, the designed values have been computed by using the data given in the report. Regarding the DC designed depth, equation (4.1) was in use. The empirical coefficients for the specific soil conditions came out in the paper to be 0.7 in trial area 1, while in the other areas 0.6. Table 5-12 presents the designed values for DC method.

Table 5-12: Designed values for DC method

Trial area	Weight of tamper *(ton)*	Drop height *(m)*	Correlation coefficient	Treatment depth *(m)*
1	17.0 25.0	9.0 22.0	0.7 0.7	8.7 16.4
2	17.0	12.0	0.6	8.6
3	17.0	12.0	0.6	8.6
4	17.0	12.0	0.6	8.6
5	17.0	12.0	0.6	8.6

In the calculation of RIC designed depth, equations (4.2) to (4.4) was utilized. The number of blows are estimated from the reported logger data originating from the data acquisition system on-board the excavator. Table 5-13 shows the designed values for RIC method.

$$v = \sqrt{2gH_d} = \sqrt{2 \cdot 9.81 \frac{m}{s^2} \cdot 1.2\, m} = 4.85\, m/s$$

$$M = W_t \cdot v = 16\, t \cdot 4.85 \frac{m}{s} = 78\, ton\, m/s$$

$$D_i = a_Z + b_Z \log(N_d \cdot M)$$

Table 5-13: Designed values for RIC method

Trial area	Number of blows *(Pcs.)*	Treatment depth *(m)*		
		Increase in the relative density		
		40 %	*20 %*	*10 %*
1	65	4.7	5.9	7.0
2	65	4.7	5.9	7.0
3	~70	4.8	6.0	7.2
4	~70	4.8	6.0	7.2
5	70	4.8	6.0	7.2

The designed values in both Table 5-12 and Table 5-13 prove to be way too smaller than the achieved values. There may be a better compilation of the penetration tests to obtain a clearer perception of pre- and post-improvement results.

6 Assessment of Using DC or RIC

This chapter will highlight both the advantages and disadvantages of using the DC and RIC methods as well as explanations given when the methods are favourable and, in turn, more unfavourable to apply with the research questions in mind. Interpretations and analysis of the results from case histories will be presented along with references to the guidelines to prove why or, on the other hand, establish and make sure why it does not work using the methods in some cases.

6.1 Soil Conditions

After studying the case histories, both the DC and RIC methods fit to a wide range of materials, but it is primarily about non-cohesive materials. In some cases, there is some amount of fines as well as silts included. In the Jätkäsaari case it was mentions also that debris and organic stuff have been compacted, but it is still unclear what kind of effect this has on a long-term settlement.

However, the soil improvement methods work considerably better when the content of fines in sand is limited, especially when using RIC. Vink & Dijkstra (2016) point out in their text that a small dip in the resistance occurs because of some fusion of fines. The same issue can be read from the tank terminal project in Amsterdam, there Dijkstra & Nooy van der Kolff (2012) prove that the dredged sand taken from North Sea containing no fines is much preferable compared to the local sand with fine fractions up over 10 %. Kristiansen & Davies (2004) mention also about a need of excavation and replacement of the top soil layer consisting of significant fines before the compaction program could take place. Additionally, they point out also a dip in the cone resistance values at the deeper zones associated with a higher number of fine materials in the granular soil. The guidelines (Lukas, 1995) declare a soil existing of a great portion of fine materials including silts as well, may be less applicable for impact oriented compaction methods, when examining the graph in chapter 4.3. According to me, DC method can better manage an improvement of a soil with a somewhat finer fraction than RIC does. Since DC method developing shock waves from a relatively powerful impact, the water trapped in between the soil particles has easier to be expelled out. Soil particles can afterwards be rearranged into a denser state. RIC method is also very dependent on creating a hard plug that gradually progresses deeper down. This is something that is more difficult make in a saturated soil where the permeability is poor.

Anyway, there are also those projects (Kozompolis, Vettas, & Chlimintzas, 2013) with a certain amount of both fines in sand and silts, and still showing an appropriate result in degree of compaction. What separates this project from the others is that the amount of applied compacting energy that has been used is somewhat larger if compared with the other projects. In particular, when using rapid impact compaction, see Table 6-1 and Table 6-2. Usually, when saturated soils that containing a higher amount of fines is exposed by an increased number energy input, a rapid change in soil response will occur. A change from granular to pseudo-cohesive that slows down the energy transfer (Slocombe, 2005).

In none of these all case histories a clayey soil has been compacted. The guidelines (Lukas, 1995) also recommends that the fill deposits for clayey soils should be only partially saturated, situated well above the ground water level and possess a good surface drainage.

This scenario is rarely seen. Thus, the improvement of clayey soils requires often more care. Therefore, adopting stone columns, which both strengthen the weak soil and work as a drainage, in cooperation with DC has proved to be of greater success (Slocombe, 2005). The case histories seem to more or less follow the directive from the guidance of applicable soil types. In three of these seven cases, it has been concerned about loose naturally occurring soils either dredged from sea bottom or a local sand, which demonstrate a good compact to a sufficient depth even with a less amount of applied energy (Tan, 2007), (Vink & Dijkstra, 2016) and (Dijkstra & Nooy van der Kolff, 2012). In addition, there are good improvements for as well old and recent landfills as building rubbles and construction debris deposits (Havukainen, 2014).

6.2 High Water Table

The most of working sites have a soil condition that is partially saturated, if not fully saturated, with a water table that is just a few meters below the ground surface. This may make it difficult for impact oriented compaction methods to act as a soil improvement method, as it often results in an excess of build-up pore water pressure during compaction. The groundwater will rise up to the surface and enter the craters, which will cause energy loss in the implementation, as the compaction energy won't propagate deep enough to improve the soil. Thus, a high water table can act as a hydraulic barrier for energy to effectively be transferred to the soil (Lukas, 1995).

In cases where the groundwater has already appeared in the craters, a recovery is required until the majority of the excess pore water pressure is dissipated. This may last for a quite long in situations where low permeable soils are densified. A preventive measure to avoid this action is to lower the groundwater level by dewatering the area using some kind of drainage system. Another measure can be done by inserting an additional fill material to increase the distance from the ground surface to the groundwater level simultaneously providing a stable working platform for dynamic compaction equipment (Lukas, 1995). These two ways have also showed up in some of case histories.

Using an assisting drainage system in cooperation with one of the methods seems to be a good option reported by Tarawneh & Matraji (2014). In that project well-point pipes were installed in the adjacent area next to the soil improvement locations to reduce the excess pore water pressure and allowed the compaction energy to easier propagate deeper. Havukainen (2014) talks about softening of the ground in site 2 at Jätkäsaari project there a complementary fill layer was needed to lift up the surface due to the excess pore water pressure started to occur. This was something that appeared first after the third pass by using DC, while already after the first pass for RIC treatment. DC seems to handle the phenomenon easier than RIC. It usually requires a high impact force to ram the soil, so the air voids are forced out and the particles are rearranged in a denser state compaction (Slocombe, 2005). This may be better applied by using DC device.

Still, RIC method shows an appropriate result in the project reported by Vink & Dijkstra (2016) even if the water level is at the surface, see Figure 5-5. Thus, the wet conditions do not seem to affect the RIC treatment either, but in this case a 2-meter-thick stockpile material of coarser grade was placed on top of the compressible layer. An extra thick complementary fill layer could in turn reduce the depth of improvement below the layer.

Therefore an additional fill on top should be avoided as far as possible, or gradually increase the fill materials (Lukas, 1995; Slocombe, 2005). However, as it was said in the literature review, the soil particles are allowed to move into a denser position during the dissipation of pore water pressure. Thus, according to me, an occurrence of pore water pressure is not of a bad nature especially in granular soils. Furthermore, Slocombe (2005) writes also in his text that due to the dissipation and additionally the loads from the overlying soil mass, the liquefiable soil is further forced into a denser state of compaction and an increase in relative density happens quite fast.

6.3 Presence of Hard or Soft Layers

An energy-absorbing layer in the deposit can cause a huge reduction in the depth of influence. This is both established in guidelines (Lukas, 1995) and occurs in some of the case histories. It can be in terms of a hard layer above the actual compressible soil or as a soft layer situated within the improvable soil deposit. A hard layer has the effect of absorbing the compaction energy, thus the energy transfer decreases significantly before it reaches the lower compressible layers. Such a retardant layer should either be loosened or excavated before the treatment is executed, if it is not relatively thin and located near the surface. In such situations, the pounder is usually able to penetrate through the layer and deliver energy further down. If there is a soft layer within the intended treating layer, a consideration of an excavation or a stabilization by driving a coarser material into the soil should be carried out, because a very soft or organic deposit tends to absorb energy during impact oriented compacting. The lower layers are not treated to the desired extent then (Lukas, 1995; Slocombe, 2005).

Dijkstra & Nooy van der Kolff (2012) reported on trials there two different types of ground improvement methods were executed to improve the engineering properties of the underlying peat and clay layers. Treatment of the complete surface area in an overlapping grid compressing the peat and clay layers was attempted, as well as a formation of stiff columns transmitting the external loads down to the bearing layers. None of them succeeded. It was said that the compressible layer was of a very hard nature, but independently, both the DC and RIC methods should be used carefully when it comes to the presence of hard and soft layer in the formation.

Kozompolis, Vettas & Chlimintzas (2013) emphasize a similar problem within the Arabian Peninsula project using RIC within site area 3. There was a weak soil layer in the form of clay and silt found within the influence zone. Anyway, the area received a post-improvement value that was well much larger than the other RIC treated areas, probably due to a potential reflection of the compaction energy from the underlying fine-grained layer. This could also be noticed in Dijkstra & Nooy van der Kolff (2012) project. Thus, I think a presence of a hard or soft layer can be of a great benefit as long as the performance requirements ranging within appropriate values.

6.4 Degree of Compaction and Applied Energy

One of the main factor in the selection of soil improvement method is the degree of compaction as well as the productivity. The latter of them will be discussed more in detail in section 6.5. Table 6-1, Table 6-2 and Table 6-3 show a compilation of the achieved values versus the designed values of soil improvement. The designed depth of using RIC has been calculated by Oshima & Takada's (1997) equation (4.4). The result indicates a difference in the relative density by 40%, 20% and 10%. In a case of DC, Lukas (1995) equation (4.1) has been applied. The used empirical coefficient has been established using the Table 4-1 based on the soil conditions described in the paper, only in cases where it is not mentioned. The applied compaction energy has as well been displayed for each specific area to evaluate the effect of different amount of energy input.

There are two different achieved values displayed in the Table 6-1 to Table 6-3. One value demonstrates the depth of the top layer with the largest improvement and the second value, in the brackets, shares the maximum achieved depth. The achieved values are also highlighted with different colours depending on how they stand in relation to the designed value. A green colour tells when the achieved value appears to be greater than the designed value, while with a red colour recognizes when the achieved value is smaller. An orange colour means that the values stand equal to each other.

It can clearly be seen that the achieved depth is slightly greater than the designed depth basically for all case histories. This makes any of the results somehow a bit optimistic. Kozompolis, Vettas & Chlimintzas (2013) mention in their paper that the RIC treated soil has been applied by a compaction energy up to 400 and 570 ton-m/m^2 in some areas. An estimation of applying a grid spacing of 2.0 m by 2.0 m gave a cumulative number of weight around 1,900 ton-blows. If a 9-ton hydraulic hammer was used (see Figure 5-8 b) within three passes, then 60 to 70 drops per impact point per pass is required. It sounds as a lot of compaction drops for RIC treatment compared to the other case studies. Thus, the applied energy may have met a threshold energy. However, the poorly graded sand with silt consisting of a relatively huge number of fines in a saturated condition may require a low-energy input within many tamping passes (Slocombe, 2005). This may be the reason why the applied compaction energy is increased to this level. It can be stated at the same time that within a poor pre-improvement soil conditions DC method seems to be more effective than RIC method in treating the soil. Anyway, if Figure 5-10 to Figure 5-12 from RIC compaction are reviewed it is more likely to demonstrate an improvement down to a depth of only 4 to 6 m.

There are also those soil improvement projects that are executed for very loose naturally and hydraulically placed soils (Dijkstra & Nooy van der Kolff, 2012), (Tan, 2007) and (Vink & Dijkstra, 2016). The loose pre-improvement state of soil may explain the reason why such a good compaction is occurred in these projects as well as the less number of fines (Slocombe, 2005). Kristiansen & Davies (2004) display also a depth of compaction that is far below the designed value, where the deposit consisted of cohesive soils and silt zones occasionally. They point out in the paper that the zones were alternately firm and loose to compact, thus the key of the exceptional treatment depth might be occurrence of loose zones at some point, probably for case (a) see Figure 5-21.

Havukainen (2014) presents a result given by using inclinometers and static-dynamic penetration tests. Conducting a review of these quality assurance tests reveals a degree of compaction between 7.0 m and 9.0 m by using RIC, further depths could be reached where the pre-improvement seemed to be very loose. On the other hand, DC might be able to compact a somewhat larger depth.

Table 6-1: Designed and achieved treatment depth from case histories using RIC

Case History	Weight of hammer W (ton)	Drop height H (m)	Impact velocity v (m/s)	Momentum M (ton-m/s)	Number of blows N (blows)	Applied energy AE (ton-m/m^2)	Designed depth D (m) (Oshima & Takada, 1997) $\Delta D_r = 40\ \%$	$\Delta D_r = 20\ \%$	$\Delta D_r = 10\ \%$	Achieved depth[1] Largest Imprv (Max. depth) (m)
Felixstowe (UK)	16	1.2	4.85	78	45	114	4.2	5.3	6.3	**6.0 – 7.0** (8 – 10.5)
Arabian Peninsula		1.2	4.85	44						
Area 1						400 – 570[2]	5.6	6.9	8.2	**5.8** (7.0)
Area 2						70 – 250[2]	4.4	5.5	6.6	**4.2** (6.5)
Area 3						150 – 300[2]	4.7	5.8	7.0	**4.0**
Amsterdam (The Netherlands)	16	1.2	4.85	78	55	112	4.5	5.6	6.7	**5.0 – 8.0**
Dubai (UAE)	9	1.2	4.85	44	60	90	3.8	4.8	5.7	**1.5 – 3.5** (5.0)
British Colombia (Canada)	7.5	1.2	4.85	36	60	195	3.5	4.5	5.4	
Area 1										**8.5**
Area 2										**5.5**
Area 3										**6.0**

[1] the column provides two achieved value; the depth of the top layer with largest improvement (usually through an own analysis of the result) and the maximum achieved depth (usually mentioned in the text)

[2] an estimated square drop spacing of 2 m has been used. This gives a cumulative number of the weight (including all blows for all passes); Area 1: 1900 ton-blows, Area 2: 833 ton-blows, Area 3: 1000 ton-blows

Table 6-2: Designed and achieved treatment depth from case histories using RIC

Case History	Weight of hammer W (ton)	Drop height H (m)	Impact velocity v (m/s)	Momentum M (ton-m/s)	Number of blows N (blows)	Applied energy AE (ton-m/m^2)	Designed depth D (m) (Oshima & Takada, 1997) $\Delta D_r = 40\ \%$	 $\Delta D_r = 20\ \%$	 $\Delta D_r = 10\ \%$	Achieved depth[1] Largest Imprv (Max. depth) (m)
Jätkäsaari (Finland)	16	1.2	4.85	78						
Area 1					65	200 – 312	4.7	5.9	7.0	9.4
Area 2					65	140 – 312	4.7	5.9	7.0	7.9
Area 3					~70	192 ~ 600[2]	4.8	6.0	7.2	~ 9.5
Area 4					~70	120 – 340	4.8	6.0	7.2	~ 9.0
Area 5					70	85 – 340	4.8	6.0	7.2	10.5

[1)] the column provides two achieved value; the depth of the top layer with largest improvement (usually through an own analysis of the result) and the maximum achieved depth (usually mentioned in the text)

[2)] due to the very loose soil more than 100 blows must be given within a relatively large area

Table 6-3: Designed and achieved treatment depth from case histories using DC

Case History	Weight of tamper W_t (ton)	Drop height H_d (m)	Empirical coefficient n_c	Applied energy AE (ton-m/m²)	Designed depth D_{max} (m)	Achieved depth (m)
Massachusetts (U.S.)	14.4	18.3	0.5[1]	224	8.1	**7.0 – 8.0** Max. 11
Arabian Peninsula	20.0	23.0	0.37 – 0.4[2]	350 – 400	7.9 – 8.6	**7 – 8.5**
Jätkäsaari (Finland)						
Area 1	17.0 25.0	9.0 22.0	0.7[2] 0.7[2]	294 264	8.7 16.4	**12.5**
Area 2	17.0	12.0	0.6[2]	392	8.6	**7 – 9.4**
Area 3	17.0	12.0	0.6[2]	392	8.6	**8.5 – 10**
Area 4	17.0	12.0	0.6[2]	490	8.6	**8 – 8.5**
Area 5	17.0	12.0	0.6[2]	490	8.6	**10 – 10.4**

[1)] estimated coefficient using the Table 4-1 based on the soil conditions described

[2)] is mentioned in the text

When later investigate the applied compaction energy it can be noted that a loose homogeneous naturally and hydraulically soil deposits do not require considerable amount of energy input to achieve good results. This applies to both DC and RIC treatment (Dijkstra & Nooy van der Kolff, 2012), (Tan, 2007) and (Vink & Dijkstra, 2016). A coarser graded soil treated by DC may also be densified with a bit less amount of energy considering the area 1 in Jätkäsaari case. Inclinometers showed that the depth of 12.5 m was already achieved after the four first passes using the smaller weight (Havukainen, 2014). Most probably because of the DC method is a ‘bottom-up’ method, there no further penetration from the high energy compaction phase.

According to Havukainen’s (2014) report, a large amount of energy is applied in areas 4 and 5 there the in-situ density was presumably already quite dense due to the superstructure. Thereto, area 3 required also a quite large amount of applied compaction energy before the formation could stay in a more stable level and the particles were rearranged into a denser state. It was reported that the area settled quite much in some places after the first pass. This indicates a soil in loose state that’s probably the reason why compaction energy was required.

It is remarkable to mention that the project (Tarawneh & Matraji, 2014), where the area was dewatered by using well point pipes, showed an effective result with significantly lower applied compaction energy. Finally, it can be said that generally there is a greater need for energy input when using DC method compared to RIC method.

6.5 Productivity and Economically Viability

Productivity among the alternative method is of a big impact in the choice of soil improvement technique. It reflects directly the operation time and secondarily the costs for a project. Havukainen (2014) writes in the Jätkäsaari project that production rate for DC ranges between 200 to 300 m^2 per shift for one machine. On the other hand, a productivity for DC in the soil improvement project in Dubai was estimated to a rate of around 1,000 m^2 per shift per machine. The difference may be that these jobs are performed within flat desert land without any obstacles or civilization within the nearest distances and with a slightly larger used equipment as well. However, the 29,000 m^2 large project area was completed with RIC method in three weeks. It presented a working rate of 2,500 m^2 per shift for one machine (Tarawneh & Matraji, 2014).

Dijkstra & Nooy van der Kolff (2012) also mention within the ground improvement work in Amsterdam that the RIC method could produce a working rate of around 2,000 m^2 per working day. All the excavation down to a depth of 5 to 8 meters, back filling and compaction was carried out within four months for an area of approximately 125,000 m^2. Havukainen (2014) demonstrates in Jätkäsaari case a production rate for RIC being just over 500 m^2 per working day. It should be borne in mind that these compaction works are performed in small trial areas where the work could be experienced crowded at times.

With that said, it can be seen that RIC performance is twice as effective in its kind as DC. Dubai project indicates for large areas there the performance difference may be 2.5 times better when using RIC compared to DC, which is also verified by Havukainen (2014). This is something that makes RIC significantly more effective and is of a major impact in terms of the final costs, when the required working time is shortened. Kozompolis, Vettas & Chlimintzas (2013) state in their paper that soil improvement costs developed by using RIC method are reduced by about 60% of the corresponding DC costs. A less costly and more productive device can also be noted when the evaluation of three different soil improvement methods are made in the report (Tarawneh & Matraji, 2014). Something that can cause dilemma is the soil improvement target zone and the availability of device, as mobilization costs can be high (Kozompolis, Vettas, & Chlimintzas, 2013) and (Havukainen, 2014).

6.6 Ground Vibrations

Implementation of soil improvement work in urban areas requires caution that the surrounding structures do not damage from potential ground vibrations. RIC method seems to carry out a very controlled work and is able to limit the risk of exceeding target vibration levels, where soil improvement can get close to the attentive structures. Dijkstra & Nooy van der Kolff (2012) reported on a safely done work because of a special working method. The compaction was performed towards the exposed structures leaving some loose soil constantly between the structure and impact point. Seismographs in projects (Tarawneh & Matraji, 2014) and (Kristiansen & Davies, 2004) displayed also measurements below the permissible values, for very short distances, 8.5 m and 5 m respectively. Kristiansen & Davies (2004) presented a construction of a trench to reduce the vibration transmissions, while Tarawneh & Matraji (2014) executed a work without any arrangements.

Considering the induced vibrations and deformations from DC and RIC methods. Tan (2007) and Vink & Dijkstra (2016) gave respectively a report on two almost similar scenarios of the compaction work very close to the retaining wall. RIC could do a job as close as only 40 cm from the sheet pile walls and still the deformations remained within the criteria. Tan (2007) said, on the other hand, that DC needed to keep a distance greater than 4.6 m to ensure no damages would occur. However, there are no distances to talk about for any methods.

The acceptable vibrations generated by RIC depend to a large extent on a generally lower drop height as well as a smaller imparted foot imprint. The applied compaction energy is also a significant factor of the induced vibrations, which is usually in a far greater kind for DC. However, it is not only the applied energy at once that is crucial. The cumulative applied energy is not to forget since vibrations at a compaction point tend to increase as the cumulative applied energy increases. Anyway, the frequency is very important to keep in mind when using impact oriented compaction methods. As the safe vibration level can be exceeded by a certain frequency and vibration acting together. This may cause devastating damage to a building (Lauzon, Morel, Briet, & Beaton, 2011).

7 Conclusion

These soil improvement methods are most successfully applied for loose naturally and hydraulically soils that are mostly homogeneous. Old as well as recent landfills consisting of granular soils are also yielded in a good way, but both DC and RIC seem to not work that well when the energy has to transfer from one medium to another. This is something that pretty frequent leads to additional actions at site. The content of fines is another crucial factor when performing impact oriented compaction, DC method works somehow better in those conditions than RIC.

A high water table does not prevent the methods from being used, but it may reduce the effect of compaction energy to propagate to deeper layers. However, it should be mentioned that good results were shown when water levels were reduced by means of dewatering systems, and significantly less compaction energy was applied. The applied compaction energy differs slightly from case to case, as for the depth of improvement. DC generally presents a somewhat higher average applied energy than RIC as well as a greater depth of improvement. Anyway, these variables don't progressively increase at the same rate as the other increases. They are more likely dependent on ground conditions.

It appears that DC method is more effective than RIC, when target zones are relatively thick or the soil conditions seem to be poor. On the other hand, RIC method is revealed to be very useful, when the influence zones are within a range of around five-six meters. Since a twice as good production rate compared to DC as well as less costly, the soil improvement method proves favourable. However, RIC method has also achieved greater depths by using more powerful equipment. The low vibration effects generated from RIC enable the project to benefit from the method when compacting next to nearby structures. It performs a more controlled work compared to DC.

In view of the usefulness of the thesis and what has been achieved. The industry has now the opportunity to further consider whether any possible commissioning of the RIC method would be beneficial to them, or still prefer the DC method. It can be determined if the RIC method is of a better use compared to the already used DC method. According to me, the industry could benefit from the RIC method when the target zones are within reach, but as much else, it requires a frequent use to repay the user. Therefore, relatively large-scale projects with an extensive duration time are highly appreciated. As a transportation the devices long distances along the roads is not profitable. Hence, an insufficient availability of device may be costly, because the mobilization costs can be perceived expensive. This assumes that the projects are more concentrated within one area. The DC method has the advantage that it does not require any major arrangements in use, and most construction sites are equipped with a required crawler crane all the time.

The evaluation of the methods has been carried out through analyses and interpretations of typical soil improvement projects worldwide. In further studies, it would be possible to focus more on a specific business area, e.g. in the Nordic countries, and look more into the performance in those soil conditions. A participation in any executed compaction trials might have provided the work with completely different data. There could be the possibility of being involved and influencing the starting points for the work. This would probably have given a different perspective on the work too.

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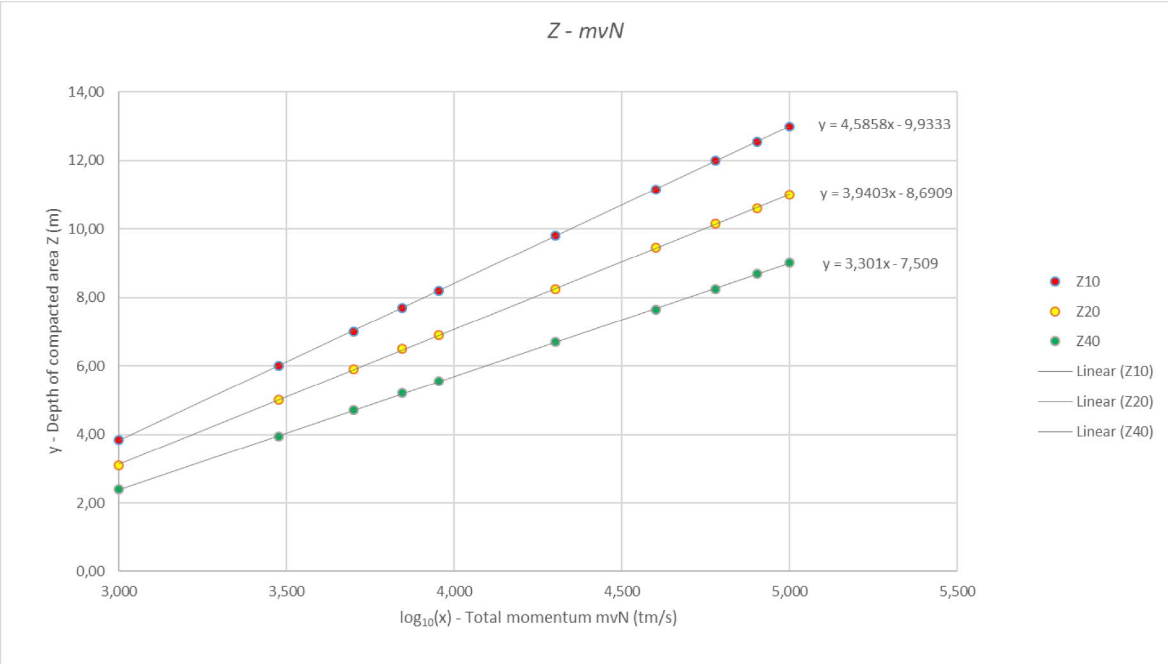
Depth correlation coefficients

The correlation coefficients have been calculated using the initial data from Oshima & Takada's (1997) charts (*Fig. 10 Relation between compacted area and total momentum*) p. 1644. Several points (x and y values) have been read from the chart, then plotted in a new table. Using the table values, a new chart have displayed in Microsoft Excel, which created a linear equation. From the linear equation, depth correlation coefficients a_Z and b_Z can be read. This has been done for respective relative density increases.

Z_{10}		
x	$log_{10}(x)$	y
1000	3,000	3,85
3000	3,477	6,00
5000	3,699	7,00
7000	3,845	7,70
9000	3,954	8,20
20000	4,301	9,80
40000	4,602	11,15
60000	4,778	12,00
80000	4,903	12,55
100000	5,000	13,00

Z_{20}		
x	$log_{10}(x)$	y
1000	3,000	3,10
3000	3,477	5,00
5000	3,699	5,90
7000	3,845	6,50
9000	3,954	6,90
20000	4,301	8,25
40000	4,602	9,45
60000	4,778	10,15
80000	4,903	10,60
100000	5,000	11,00

Z_{40}		
x	$log_{10}(x)$	y
1000	3,000	2,40
3000	3,477	3,95
5000	3,699	4,70
7000	3,845	5,20
9000	3,954	5,55
20000	4,301	6,70
40000	4,602	7,65
60000	4,778	8,25
80000	4,903	8,70
100000	5,000	9,00



Depth Coefficients	a_Z	b_Z
$\Delta D_r = 40\ \%$	-7,509	3,301
$\Delta D_r = 20\ \%$	-8,691	3,94
$\Delta D_r = 10\ \%$	-9,933	4,586

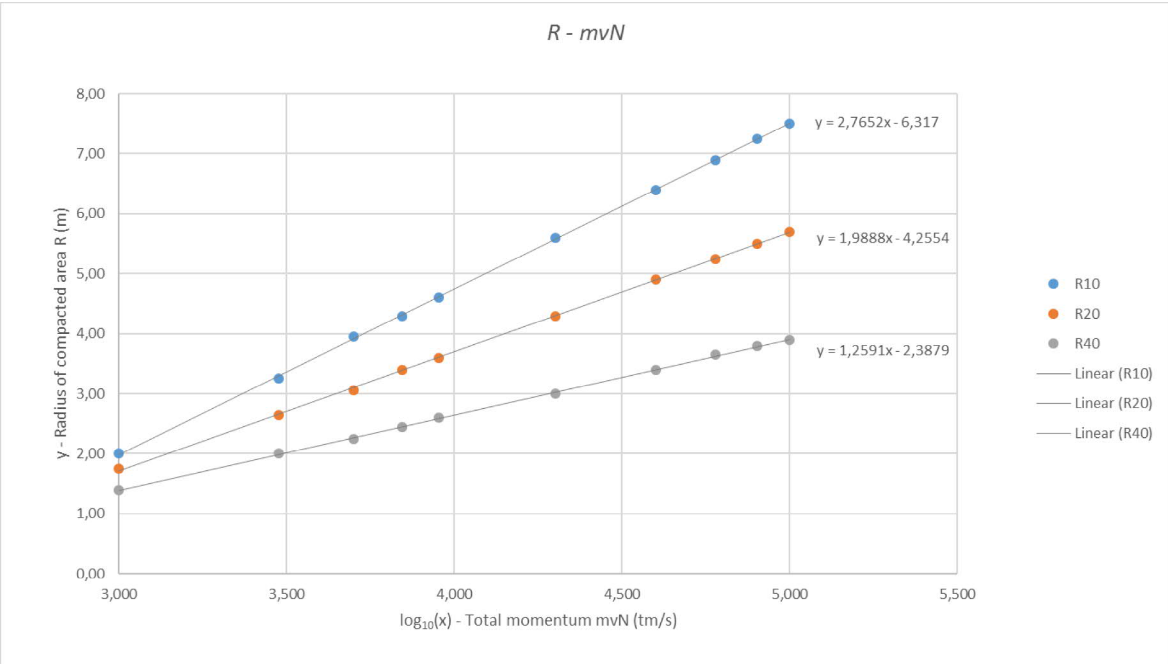
Radial correlation coefficients

The correlation coefficients have been calculated using the initial data from Oshima & Takada's (1997) charts (*Fig. 10 Relation between compacted area and total momentum*) p. 1644. Several points (x and y values) have been read from the chart, then plotted in a new table. Using the table values, a new chart have displayed in Microsoft Excel, which created a linear equation. From the linear equation, radial correlation coefficients a_R and b_R can be read. This has been done for respective relative density increases.

R_{10}		
x	$\log_{10}(x)$	y
1000	3,000	2,00
3000	3,477	3,25
5000	3,699	3,95
7000	3,845	4,30
9000	3,954	4,60
20000	4,301	5,60
40000	4,602	6,40
60000	4,778	6,90
80000	4,903	7,25
100000	5,000	7,50

R_{20}		
x	$\log_{10}(x)$	y
1000	3,000	1,75
3000	3,477	2,65
5000	3,699	3,05
7000	3,845	3,40
9000	3,954	3,60
20000	4,301	4,30
40000	4,602	4,90
60000	4,778	5,25
80000	4,903	5,50
100000	5,000	5,70

R_{40}		
x	$\log_{10}(x)$	y
1000	3,000	1,40
3000	3,477	2,00
5000	3,699	2,25
7000	3,845	2,45
9000	3,954	2,60
20000	4,301	3,00
40000	4,602	3,40
60000	4,778	3,65
80000	4,903	3,80
100000	5,000	3,90



Radial Coefficients	a_R	b_R
$\Delta D_r = 40\,\%$	-2,388	1,259
$\Delta D_r = 20\,\%$	-4,255	1,989
$\Delta D_r = 10\,\%$	-6,317	2,765

Inclinometer measurements

The lateral displacements have measured by inclinometer in trial area 1 and 2. Figure 1 and Figure 2 show the measurements from site 1 for RIC and DC respectively, while Figure 3 and Figure 4 show the measurements from site 2.

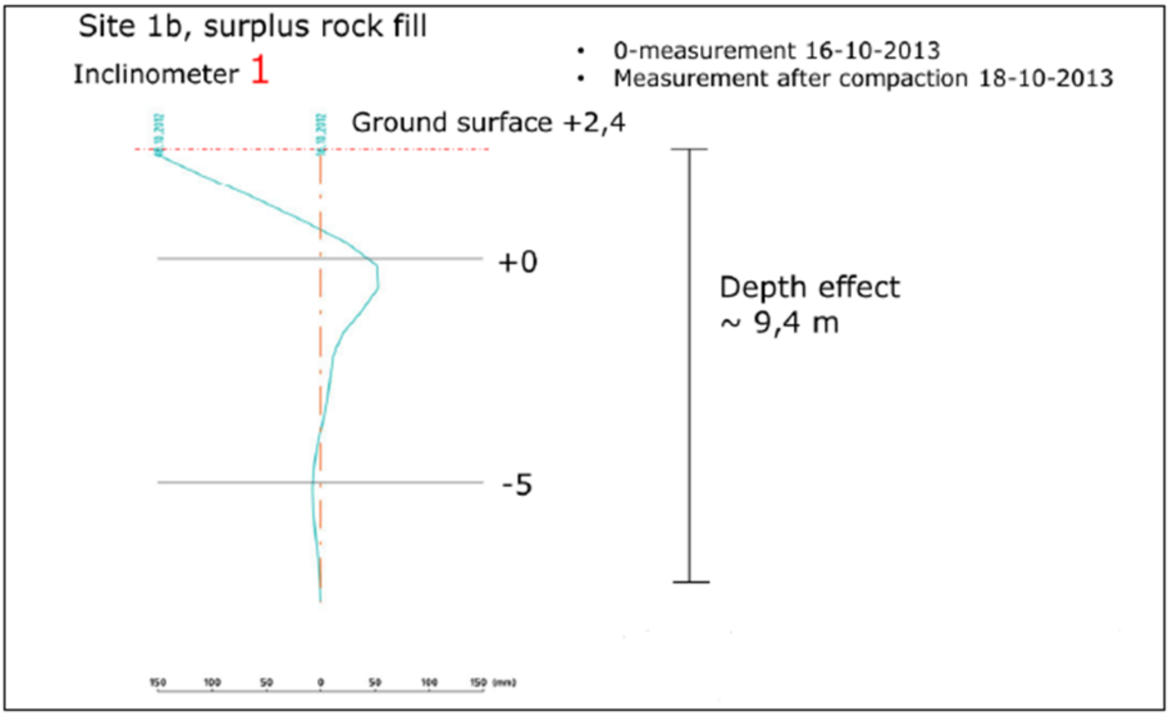


Figure 1: RIC compaction from trial site 1b.

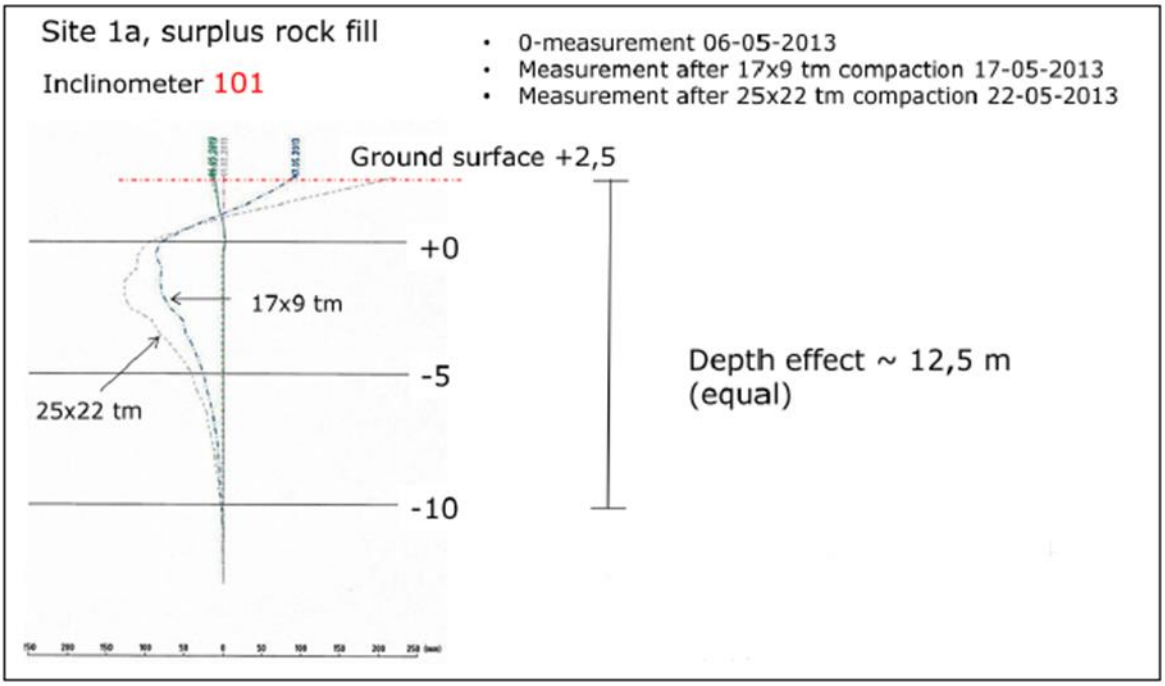


Figure 2: DC compaction from trial site 1a.

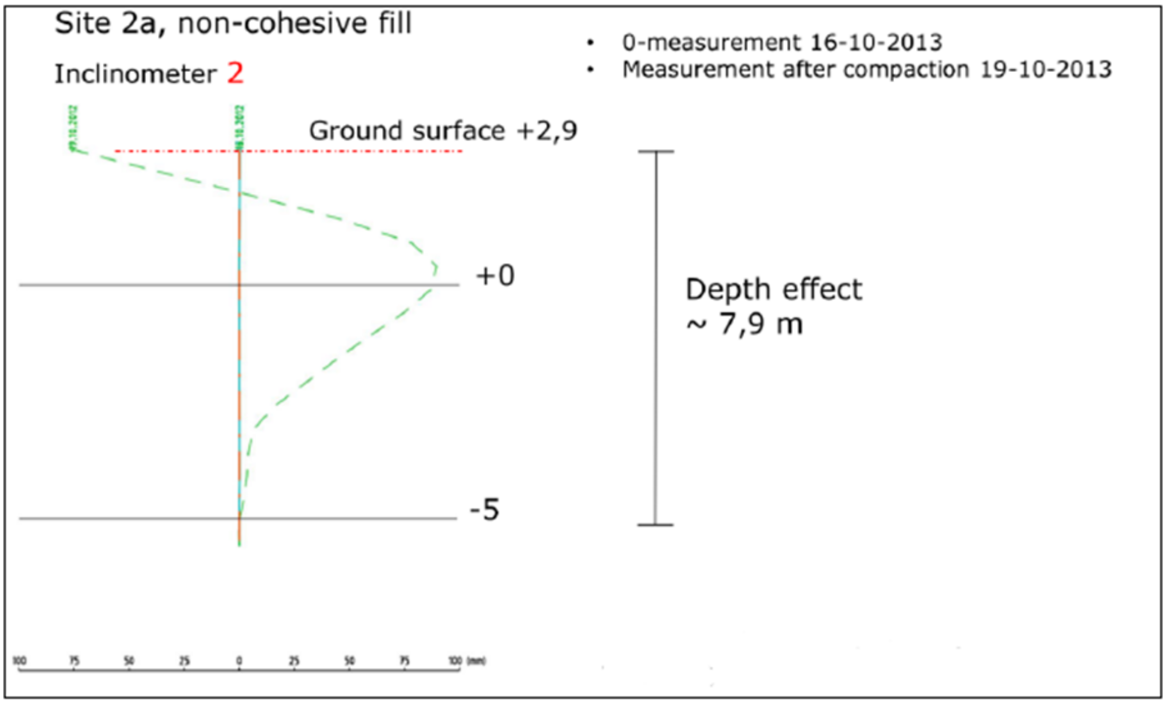


Figure 3: RIC compaction from trial site 2a.

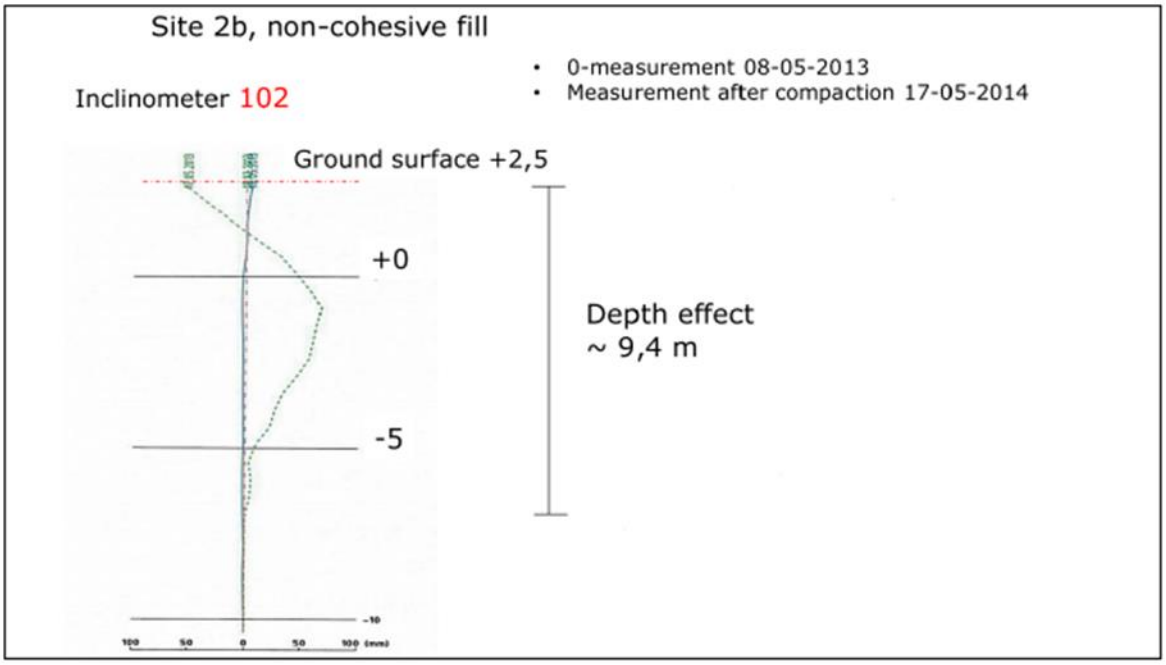


Figure 4: DC compaction from trial site 2b.

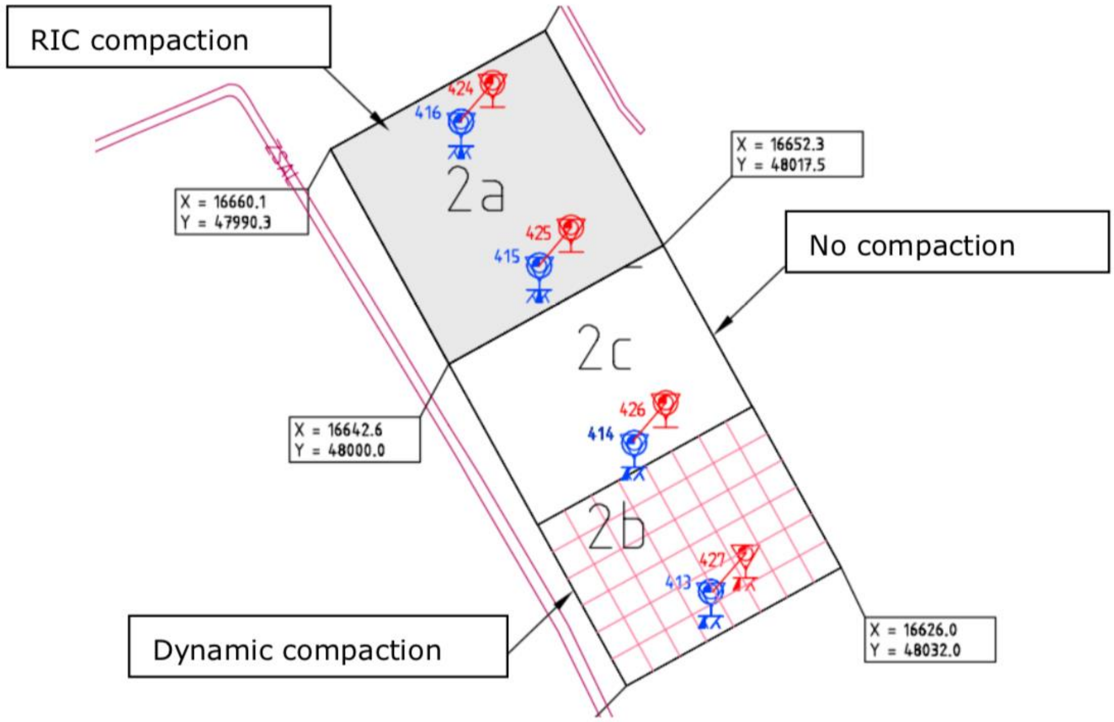
Figure 1 to Figure 4 are taken from Havukainen's (2014, pp. 34-35) report.

Static-dynamic penetration tests

Static-dynamic penetration tests (soundings) have been executed at trial area 2 to 5. Pre- and post-improvement soundings are made to find out the degree of compaction by comparison of the resistance values. The pages below demonstrate the pre- and post-resistance diagrams from soundings. Each site presents four different tests evenly distributed over the site area. Pre-resistance values are shown with blue color and post-resistance values with red color. The estimated depth has in turn been drawn with a green dash line in each diagram by Havukainen (2014).

All resistance diagrams are taken from Havukainen’s (2014, pp. 37-40) report.

Before compaction (blue) / After compaction (red)



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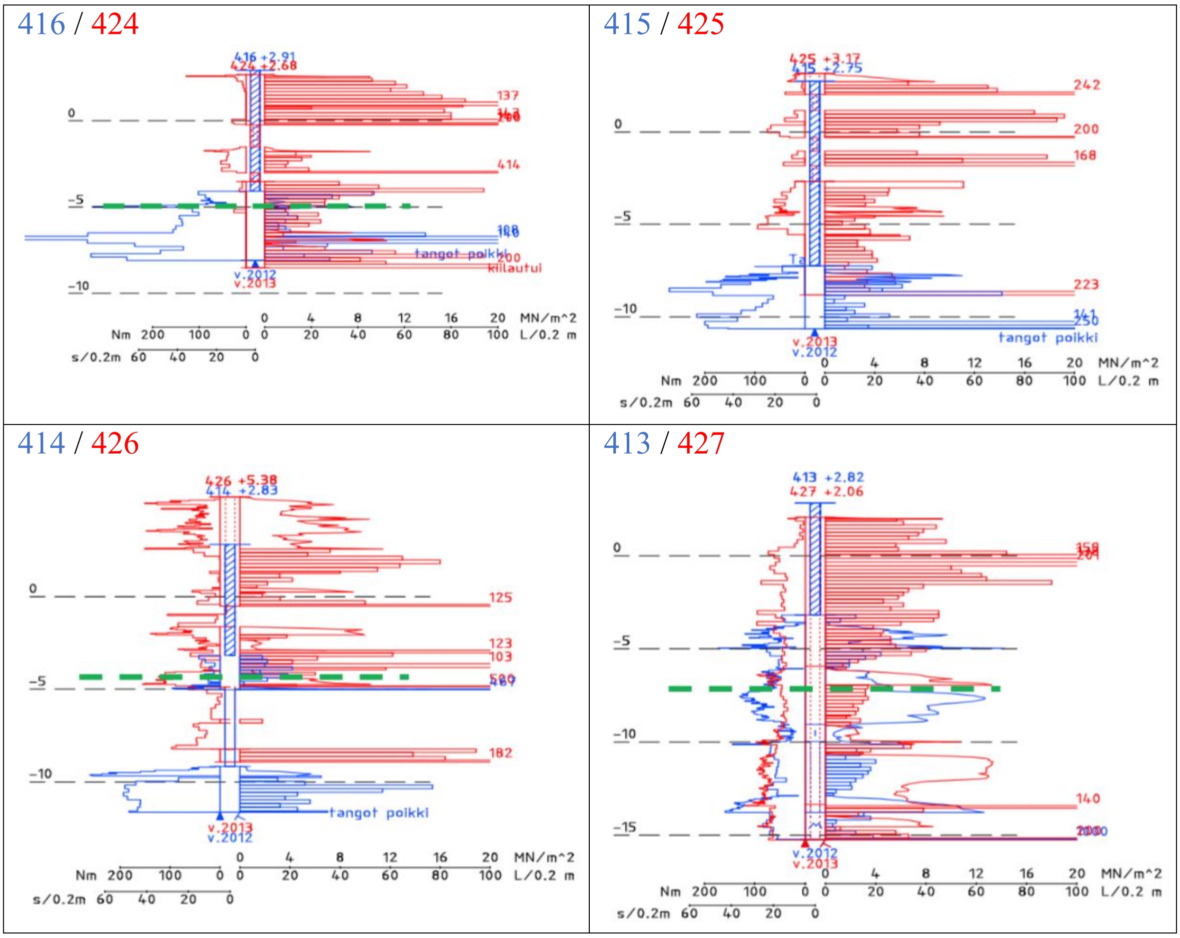

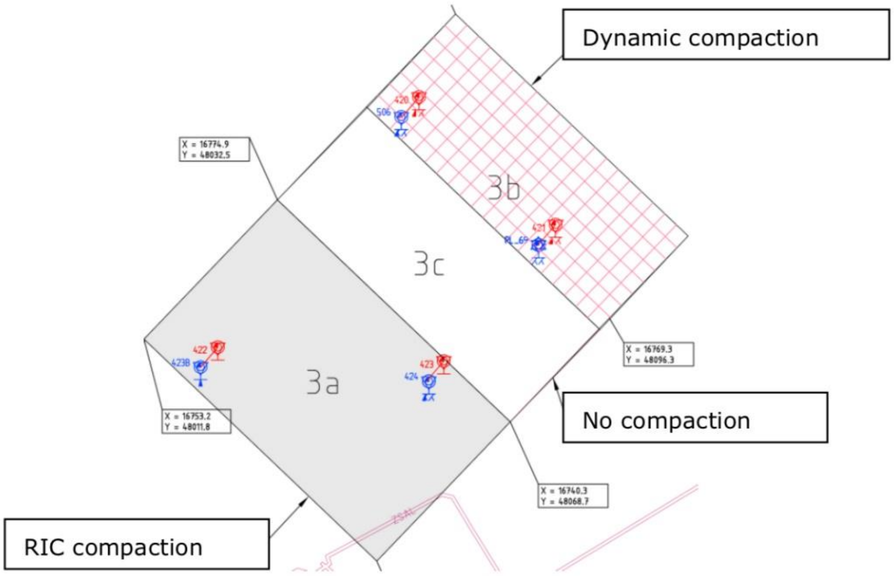

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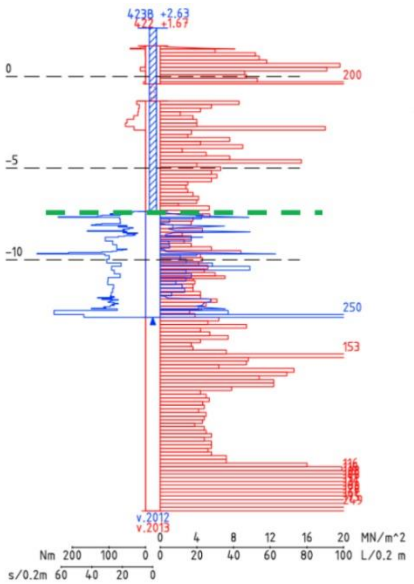

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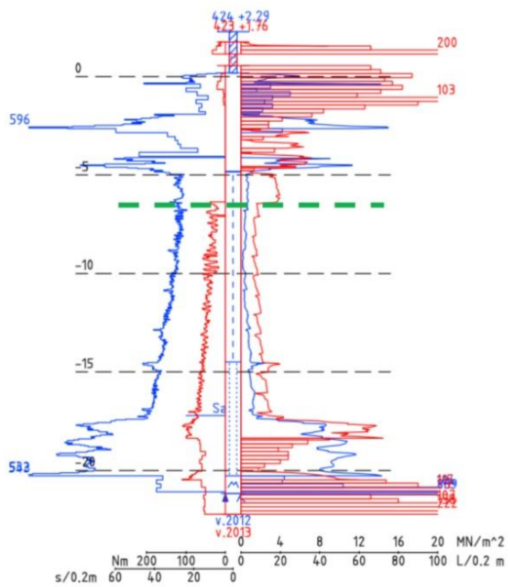

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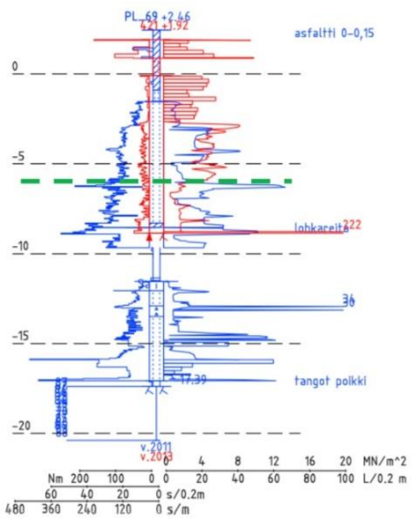

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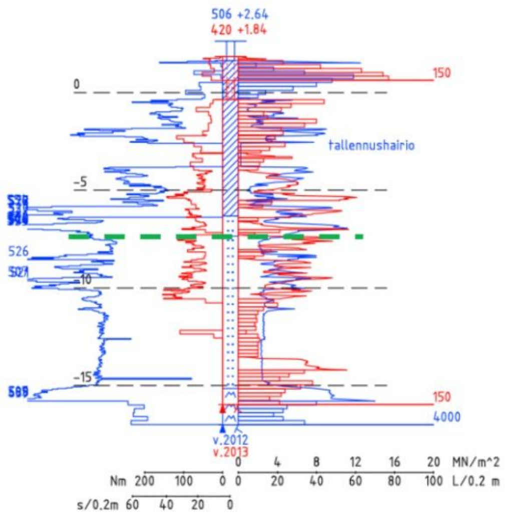

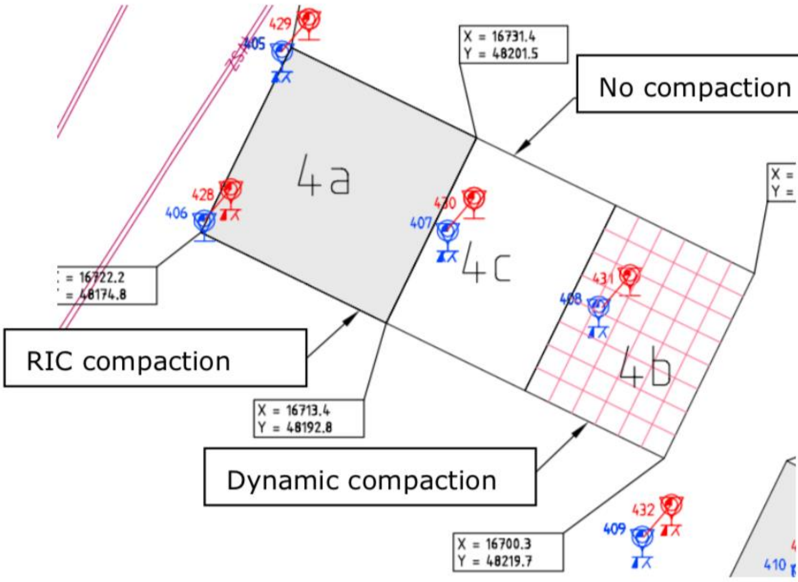

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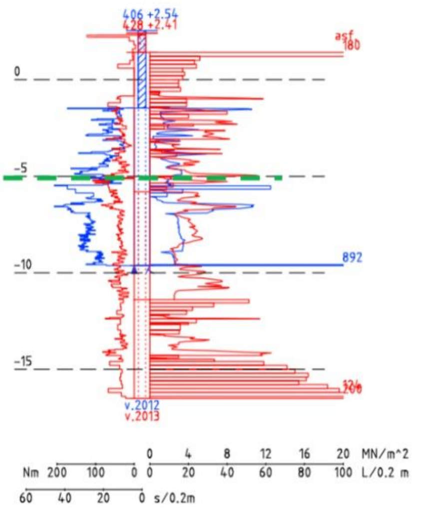

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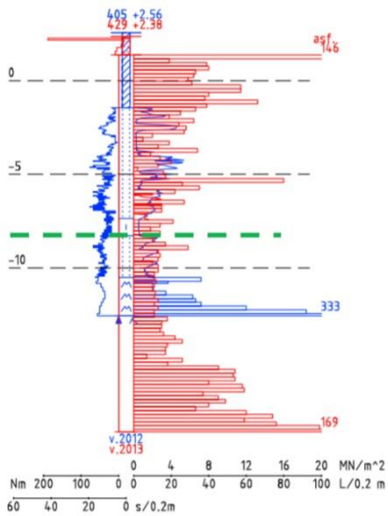

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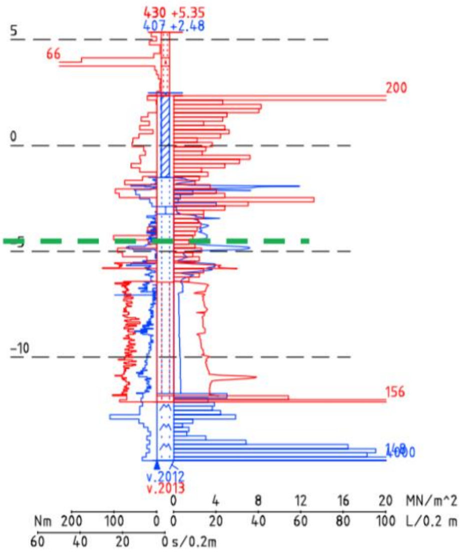

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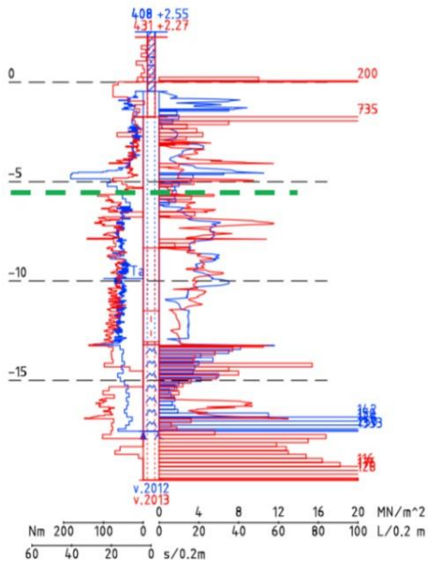

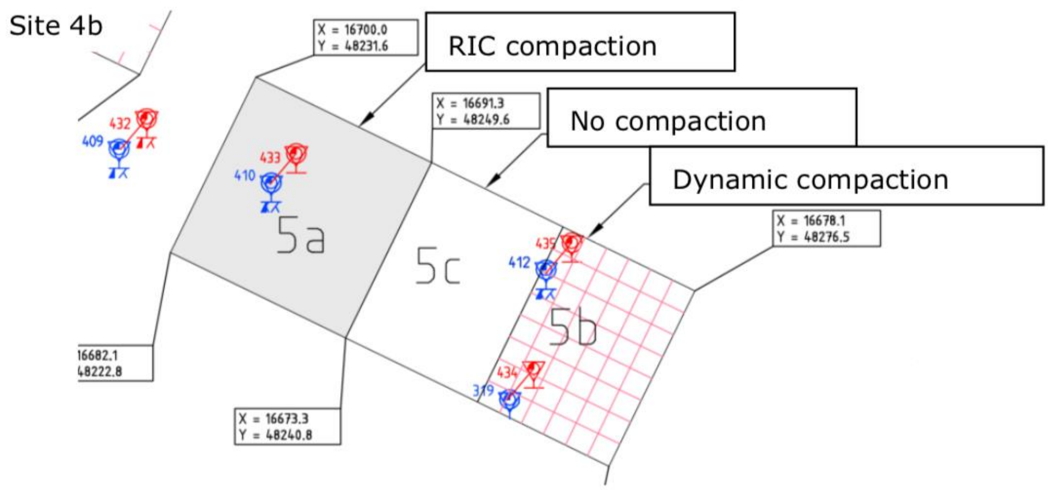

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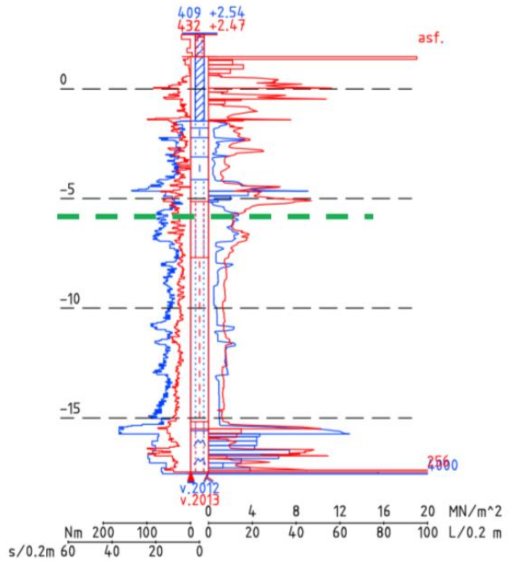

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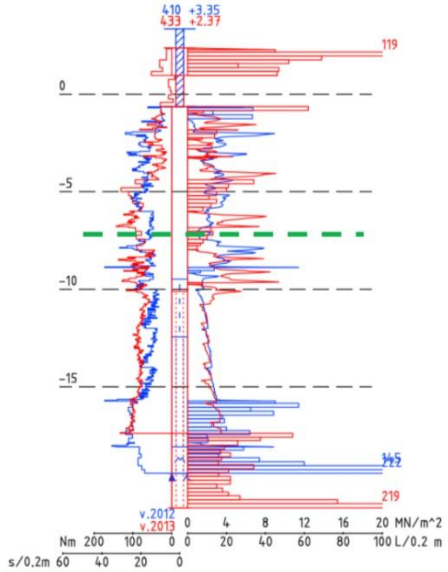

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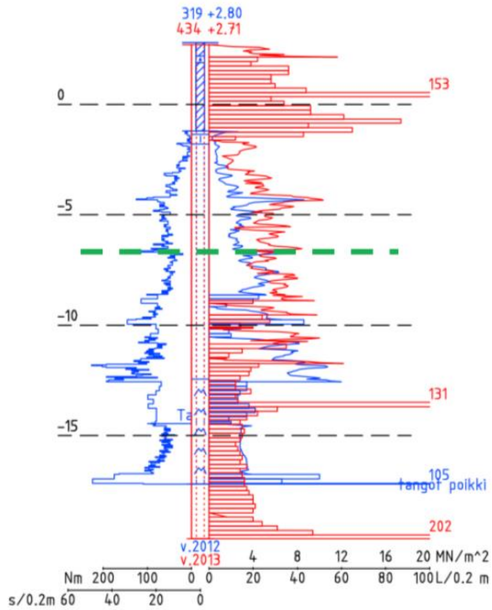

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