Framework for Field-Based Annular Pressure Prediction in Horizontal Directional Drilling

by

Inshik Park

A thesis submitted in partial fulfillment of the requirements for the degree of

Master of Science

in
Civil (Cross-disciplinary)

Department of Civil and Environmental Engineering
University of Alberta

Abstract

There have been attempts by many researchers over the years to improve the prediction of Inadvertent Return (IR) during Horizontal Directional Drilling (HDD). An analytical model of the maximum allowable mud pressure during HDD, P_{max}, has been commonly referred as "Delft method" and it has been broadly used among the practitioners over the past thirty years. Since the Delft method is generally accepted for HDD designs, most of the research has focused on improving the accuracy of estimated P_{max} compared to the actual annular pressure measured during the field operation. Despite the advantages with Delft method for its simplicity in procedure of estimating P_{max} and its logical explanation on connecting the cavity expansion theory to the HDD borehole stability; its tendency of overprediction have been reported by multiple researchers. Until lately, researchers mainly focused on reducing the size of plastic zone as an application of the factor of safety, or applying additional factor of safety as 1.5 or 2.0 on top of the calculated P_{max}. However, instead of merely applying a vague number for the factor of safety, it was thought to be more logical and efficient to suggest an appropriate factor of safety, corresponding to the risk of the operation. Depending on the depth of the borehole, surrounding soil type, sensitivity of project, etc., the factor to ensure the safe operation might vary. Moreover, accuracy of P_{max} estimated with Delft method is significantly affected by its input geotechnical parameters: c, S_u, φ, and G; however, determination of the geotechnical parameters with current practice of using SPT often poses challenges. Even though SPT provides N-value that indexes in-situ characteristics of soil, it does not include direct measurements of the geotechnical parameters; therefore, error from approximation of geotechnical parameters may lead into inaccuracy of estimation of P_{max}.

To conserve the preference of SPT and Delft method among the HDD industry and improve the accuracy of estimation of P_{max}, a design guideline for SPT-based P_{max} prediction with a framework for the factor of safety was established and introduced in this thesis. Correlations between the N-value and geotechnical parameters: c, S_u, φ, and G, which were found from multiple literatures, were evaluated and introduced into Delft equation with a conservative margin of error; moreover, resulting algorithm of the SPT-based method was also presented as a flowchart. The framework for the factor of safety was set up and specified for each possible scenario of HDD operation to allow designers to predict the limiting pressure with factors of safety tailored to the specific project. This proposed guideline was validated with the hydrofracture pressure measurements, those were provided from laboratory experiments of Queen's university and case studies of the actual HDD operations.

Acknowledgement

I would like to express my deepest and sincerest appreciation to my supervisor, Dr. Alireza Bayat, for allowing me to have a great opportunity to work on this very interesting and practical research topic. His guidance, patience, encouragement, and financial support allowed me to pursue this M.Sc. program successfully.

I would like to thank to the post-doctoral fellows in our research group at the University of Alberta: Dr. Sheng Huang and Dr. Chao Kang, for their academic suggestions, and constructive criticisms during the progress of my research. I would also like to thank our Research Coordinator at CETT, Lana Gutwin, who puts great amount of effort for coordination of all the activities within research group, and review of all the technical writing.

Finally, I would like to thank to my parents and friends for their support and encouragement throughout my life.

Table of Contents

List of Tables

List of Figures

1. Introduction

1.1. Background Information

Horizontal Directional Drilling (HDD) is a commonly used trenchless technique for installations of pipelines or other utility infrastructure without excavating traditional open-cut trenches. Instead of excavating trench for installation path, an underground tunnel is bored during HDD. During the HDD operation, drilling mud must be supplied with application of sufficient amount of pressure for cooling of borehole assembly, maintaining the borehole stability and transportation of cuttings. However, if this pressure exceeds the strength capacity of surrounding soils, the HDD bore fails and hydrofracture occurs. This hydrofracture is problematic and it should be avoided; therefore, estimation of the maximum allowable mud pressure is important for HDD design. For estimation of the maximum allowable mud pressure, multiple researchers (Luger and Hergarden, 1988; Yu and Houlsby, 1991; Xia, 2009; and others) developed models based on the cavity expansion theory. The Delft method of Luger and Hergarden (1988) has gained popularity among practitioners over past thirty years and it has become the most commonly used method; nevertheless, the accuracy of the Delft method for estimation of the maximum allowable mud pressure has been questioned by multiple researchers (Keulen, 2001; Elwood, 2008; Xia, 2009; Staheli et al., 2010; Rostami, 2017; Goerz et al., 2019; etc.). There has been attempts to improve the Delft method by redefining the size of plastic zone (van Brussel and Hergarden, 1997; NEN, 2003; Staheli et al., 2010; Rostami, 2017; Goerz et al., 2019); however, significant relationship between the maximum allowable mud pressure and size of plastic zone has not been found. Another source of inaccuracy of estimated maximum allowable mud pressure are the input geotechnical parameters of Delft method. Since the site investigation programs of typical HDD projects are proceeded with basic assessment, such as SPT, practitioners are required to make empirical estimations of geotechnical parameters, which may be less accurate compared to the measurements from actual experiments. Currently, practitioners rely on their own design experiences to improve the estimation with Delft method; however, as there is no firm guideline established related to this topic, estimation of maximum allowable mud pressure remains unsystematic and subjective. Moreover, the current method of estimation of maximum allowable mud pressure does not include the concept of risk involved with each project.

1.2. Research Objective

The main objectives of this thesis are listed as follows:

Objective 1: Develop a direct and systematic procedure for estimating maximum allowable mud pressure in HDD, using Delft method and SPT results (referred to as SPT-based method).

Objective 2: Validate the newly developed SPT-based method with measurements of hydrofracture pressure. Based on the comparisons between the estimation and the actual measurements, find an appropriate value for the factor of safety for the Delft method.

Objective 3: Propose a new conceptual framework for the factor of safety of Delft method, which takes into account the various levels of hydrofracture risks involved with different HDD bore conditions. Determine which parameters have a significant effect on the risks of hydrofracture; furthermore, adopt the determined risk parameters to set up the newly proposed framework of factor of safety for the estimation of maximum allowable mud pressure. Lastly, determine the values of safety factor for the various bore conditions.

Objective 4: Validate the Delft method using the newly developed framework of factor of safety using the measurements of hydrofracture pressure.

1.3. Methodology

For development of SPT-based method, correlations between the N-values and field/laboratory-based geotechnical parameters: c, S_u, φ, and G, were reviewed and compared. After the SPT-based method was categorized into drained and undrained soil models, the selected correlation models were adopted for the definitions of geotechnical parameters of Delft method. Once the SPT-based method was developed, its algorithm was also presented in a flow chart form. For validation of SPT-based method, data of measurements of hydrofracture pressure was collected from multiple literatures (Luger and Hergarden, 1988; Keulen, 2001; Xia, 2009; Staheli et al., 2010; and Lan and Moore, 2018). Estimations of maximum allowable mud pressure were made based on the provided information of hydrofracture occurrences, and they were compared with the measurements of hydrofracture pressure. By taking the ratio between the estimations of maximum allowable mud pressure and measurements of hydrofracture pressure, factor of safety for the SPT-based Delft method was found.

For development of framework of factor of safety (FOS), input parameters of Delft method were carefully reviewed to distinguish between the sensitive and insensitive parameters. SPT-based method was adopted for definitions of geotechnical parameters: c, S_u, φ, and G, so the strength of soil of FOS framework could be expressed in terms of N-value. By assigning N-value and cover depth as the risk variables, the FOS framework could be outlined. For determining the values of safety factors for the framework of each bore condition, currently available factored Delft methods were adopted. From comparisons between the estimated values of the maximum allowable mud pressure of Original Delft method and factored Delft methods, values of safety factors for the framework of each bore condition were found. Using the data on measurements of hydrofracture pressure collected from multiple literature sources (Luger and Hergarden, 1988; Keulen, 2001; Xia, 2009; Staheli et al., 2010; and Lan and Moore, 2018), the FOS framework developed in this work was validated.

1.4. Outline of Thesis

This thesis has the following structure:

Chapter 1: Introduction

Background information, objective, and methodology of the research are introduced, along with the structure of the thesis.

Chapter 2: Literature Review

Literature related to HDD, hydrofracture, cavity expansion theories, site investigation methods, and correlations between the geotechnical parameters and N-value are introduced in Chapter 2.

Chapter 3: Prediction of Annular Pressure Limits using Parameters from Standard Penetration Test

A newly developed SPT-based method for determination of maximum annular pressure in HDD is introduced and validated. Based on the validation work, a factor of safety for the SPT-based method is suggested.

Chapter 4: Factor of Safety for Annular Pressure Design in Horizontal Directional Drilling

A newly proposed concept about a framework for determining a safety factor for the Delft method was introduced with validation. Emphasis on the consequences of hydrofracture is also included.

Chapter 5: Conclusions and Future Works

A summary of the major conclusions and future work to be done in continuation of the research are presented.

REFERENCES

Elwood, D.E.Y., (2008), "Hydraulic Fracture Experiments in a Frictional Material and Approximation for Maximum Allowable Mud Pressure," M.Sc. Thesis, Queens University, Kingston, Ontario, Canada, July 2008

Goerz, S., Boelhouwer, N., and Taylor, J., (2019), "A New Perspective in Hydrofracture Analysis," *Proceedings of 2019 No-Dig Conference*, Chicago, Illinois, March 17-20, 2019

Keulen, B., (2001), "Maximum Allowable Pressures During Horizontal Directional Drillings Focused on Sand," Master thesis, Delft University of Technology, Delft, Netherlands, September 2001

Lan, H., and Moore, I.D., (2018), "Practical Criteria for Borehole Instability in Sand during Horizontal Directional Drilling," *Proceedings of 2018 No – Dig Conference*, Palm Springs, California, March 20-24, 2018

Luger, H.J., and Hergarden, H.J.A.M., (1988), "Directional Drilling in Soft Soil: Influence of Mud Pressure," International Society of Trenchless Technology, *No-Dig Conference*, Washington, D.C., October 16-19, 1988

Rostami, A., (2017), "Prediction and Evaluation of Annular Pressure in Horizontal Directional Drilling," Ph.D. Thesis, University of Alberta, Edmonton, Alberta, Canada, April 2017

Staheli, K., Price, C.G., and Wetter, L., (2010), "Effectiveness of Hydrofracture Prediction for HDD Design," *Proceedings of 2010 No - Dig Conference*, Chicago, Illinois, May 2-7, 2010

Xia, H., (2009), “Investigation of Maximum Mud Pressure within Sand and Clay during Horizontal Directional Drilling,” Ph.D. Thesis, Queens University, Kingston, Ontario, Canada, January 2009

Yu, H.S., and Houlsby, G.T., (1991), “Finite Cavity Expansion in Dilatant Soils: Loading Analysis,” *Geotechnique*, 41, No. 2, pp. 173-183.

2. Literature Review

2.1. HDD and Hydrofracture

Horizontal Directional Drilling (HDD) is a trenchless technique, which has been often considered for installations of pipelines or other utility infrastructures under surface obstructions, such as watercourses, wetlands, mountainous terrains, roads, railways, congested urban areas, etc. For those challenging locations, drilling underground tunnels with HDD method can be more efficient choice of creating installation paths, compared to excavating trenches with traditional open cut method. During the HDD operation, it is essential to apply sufficient amount of pressure in drilling mud to maintain borehole stability and adequate cutting removal. Minimum requirement of mud pressure varies along the bore path and it has increasing tendency as drill bit advances deeper and further from initial entry location (This tendency can be explained by rheological models introduced by: Baroid, 1998; API, 2009); therefore, it is important to estimate the required mud pressure accurately, which is a crucial component of the pump selection. If the applied mud pressure exceeds the maximum capacity of surrounding soil that holds up the borehole wall together, fracture failure will occur and drilling mud will begin to escape through the resulting crack openings. This type of failure is known as "Hydrofracture" (or "Frac-out") and it can lead the HDD operation into further problematic scenarios: leakage of drilling mud reaching up to ground surface, escaped drilling mud entering into underground aquifer or joining the groundwater flow of drainage layer, loss of circulation and pressure of drilling mud, and others. Leakage of drilling mud can cause pollution in water bodies, which is a serious environmental concern; moreover, it can cause heave-induced damages to roads, railways, infrastructures in urban areas, etc. Loss of circulation and pressure of drilling mud may delay the project, since the drilling operation cannot be continued without re-gaining these two critical components. For all the consequences of hydrofracture occurrences, remediation and cleaning must be done, which will increase significant amount of time and budget of the HDD projects; however, some severe environmental damages might not be recoverable, despite best efforts. Therefore, to avoid such problematic consequences, it is important to prevent hydrofracture occurrences by knowing the accurate maximum allowable mud pressure, P_{max}, and maintaining the required mud pressure below the P_{max} during HDD operations.

2.2. Estimation of Maximum Allowable Mud Pressure of HDD using Delft Method

2.2.1. Introduction of Delft Method

For the estimation of P_{max}, multiple analytical models were created based on the cavity expansion theory and these were proposed as design methods through further verifications and validations: the Delft Method (Luger and Hergarden, 1988), Yu and Houlsby's method (Yu and Houlsby, 1991), NEN 3650 Method (NEN, 2006), New Orleans Method (USACE, 2007), Queen's Method (Xia, 2009), etc. Since Luger and Hergarden introduced Delft method at the 1988 International Society for Trenchless Technology No-Dig Conference, it has become the most commonly used P_{max} prediction tool among the HDD industry over the past thirty years. The US Army Corps of Engineers (USACE) and the Royal Netherlands Standardization Institute (NEN) adopted Delft method with minor modifications and established design standards for New Orleans District (USACE, 2007) and Netherlands (NEN, 2006). Some researchers (Yu and Houlsby, 1991; Xia, 2009) developed P_{max} prediction methods for underground cavities with specific soil conditions: Yu and Houlsby's method for dilatant soils, and Queen's method for clays under anisotropic initial ground stress, and these methods are definitely valuable tools as references; however, they are not as widely practiced as Delft method to date. For most of the HDD projects, Delft method still remains as a standard practice for prediction of P_{max}. The original Delft method developed by Luger and Hergarden (1988) defines P_{max} as presented in Equation [2.1].

$$P_{max} = u + [\sigma'_0(1 + \sin\varphi) + c\cos\varphi + c\cot\varphi] \times \left[\left(\frac{R_0}{R_{p,max}}\right)^2 + \frac{\sigma'_0 \sin\varphi + c\cos\varphi}{G}\right]^{\frac{-\sin\varphi}{1+\sin\varphi}} - c\cot\varphi \qquad [2.1]$$

where u is initial in-situ pore pressure, σ'_0 is initial effective stress, φ is internal friction angle, c is cohesion, R_0 is initial radius of the hole, $R_{p,max}$ is maximum allowable radius, and G is shear modulus.

When Luger and Hergarden introduced the Delft method in 1988, they included two criteria for determination of P_{max}:

1) $R_{p,max}$ should be less than safe radius
2) P_{max} should be less than 90 percent of P_{lim}

Luger and Hergarden (1988) intended to keep the plastic zone within a safe radius, which they did not clearly define. In their example calculation of a pipeline crossing project for Windaassloot canal in Netherlands, Luger and Hergarden (1988) used cover depth, H, as the safe radius for the first criteria. They suggested keeping P_{max} less than 90 percent of the limit pressure, P_{lim}. As well-known from cavity expansion theories, P_{lim} is the P_{max} associated with $R_{p,max}$ approaching infinity. In their example calculation, which is the same as the one for the first criteria, they back-calculated $R_{p,max}$ with 90 percent of P_{lim}, and compared it with H. They did not clearly explain about the reason behind the selection of 90 percent of P_{lim}; however, their intention could be clearly understood as wanting to prevent P_{max} becoming excessive, so the extent of the plastic zone could be limited.

2.2.2. Suggestion of Delft Geotechnics for the CPAR Program

Until the late 1990s, US Army Corps of Engineers (USACE) did not have any standard guideline for installation of pipelines with the HDD method. The absence of guidelines resulted in great variation in permitting policies, and for some districts in US, HDD was strictly prohibited. Since HDD was known to be beneficial with cost-effectiveness and its application has been accepted by oil and gas industry all over the world, USACE was motivated to conduct the Construction Productivity Advancement Research (CPAR) program to develop the recommended guidelines for pipeline installation using HDD (Staheli et al., 1998). To support the CPAR program, van Brussel and Hergarden of Delft Geotechnics prepared a report about determination of minimum and maximum mud pressure during HDD. For the prediction of P_{max}, van Brussel and Hergarden (1997) suggested the Delft method proposed by Luger and Hergarden (1988) with application of factor of safety on $R_{p,max}$. Based on the construction requirements of NEN 3651, van Brussel and Hergarden (1997) suggested $R_{p,max}$ to be chosen as $H/2$ for clayey and peat soils, and 2/3 H for sand. Since this suggestion on $R_{p,max}$ was accepted by USACE during the CPAR program, it has become one of the most popular definitions of $R_{p,max}$ up until recently.

2.2.3. Necessity of an Additional Factor of Safety for Delft Method

It was believed the suggestion of Delft Geotechnics (van Brussel and Hergarden, 1997) on $R_{p,max}$ is capable of ensuring some margin of safety for P_{max} prediction using Delft method; therefore, this method has been broadly used by practitioners over the years. Even though the P_{max} predictions were made with application of factor of safety on $R_{p,max}$, hydrofracture events kept occurring at mud pressure below predicted values of P_{max}, and researchers began to realize that there is a tendency of overprediction in Delft method with the $R_{p,max}$ suggestion of Delft Geotechnics. From comparisons between the P_{max} predictions and actual failure pressures, researchers confirmed that the overprediction tendency is valid. Xia (2009) collected experimental results of Elwood (2008); moreover, he also obtained his own result by performing large scale hydrofracture tests in laboratory. From those results, Xia (2009) used the measured peak mud pressures to compare with P_{max} predictions using Delft method with $R_{p,max}$ suggested by Delft Geotechnics. Comparison illustrates the P_{max} predictions overestimate measured peak mud pressures by 160 to 190%; thus, Xia (2009) suggested at least 2.5 as a safety factor for the Delft method. Rostami (2017) collected laboratory test data from Elwood (2008), and Xia (2009), and field test data reported by Keulen (2001), which were originally published from Boren van Tunnels en Leidingen (BTL). From comparison, Rostami (2017) found the P_{max} predictions using Delft method with $R_{p,max}$ suggested by Delft Geotechnics overpredicted the measured failure pressures by 105%. Goerz et al. (2019) collected 30 hydrofracture occurrences from field operations and found that the Delft equation with the $R_{p,max}$ suggestion of Delft Geotechnics overpredicts the actual hydraulic fracture pressure by 63%. Being informed about this tendency of overprediction, practitioners of HDD industry started considering the application of an additional factor of safety for P_{max} prediction using the Delft method with $R_{p,max}$ suggested by Delft Geotechnics. For example, Miller and Robinson (2019) recommended a factor of safety at least 2.0 for calculation of P_{max} using the Delft method with $R_{p,max}$ suggested by Delft Geotechnics for crossings where inadvertent return (IR) can be detrimental to the project.

2.2.4. Suggestion of Staheli et al. (2010) for the Selection of $R_{p,max}$

During the sensitivity analyses for Delft equation, Staheli et al. (2010) made a significant finding about the influence of $R_{p,max}$ on P_{max} calculation. Unlike the $R_{p,max}$ suggested by Delft Geotechnics was believed to ensure the margin of safety for IR assessment, Staheli et al. (2010) found that this method actually has negligible impact on P_{max} for typical HDD projects. Applying the Delft suggested safety factors of 1.5 or 2.0 for $R_{p,max}$ was not effective; however, localizing the plastic zone around the borehole with $R_{p,max}$ less than a few feet could reduce P_{max} significantly. Staheli et al. (2010) did back-analyses using two measured pressure values causing IR of drilling mud, and they obtained values of $R_{p,max}$ to be 0.24-1.20 times the borehole diameter. Considering these findings, Staheli et al. (2010) suggested calculating P_{max} with $R_{p,max}$ less than 2-3 borehole diameter, or to apply a safety factor for P_{max} calculated with $R_{p,max}$ as H.

2.2.5. Back-Analyses of $R_{p,max}$ of Rostami (2017) and Goerz et al. (2019)

When Rostami (2017), and Goerz et al. (2019) found a tendency of overprediction of Delft method with $R_{p,max}$ suggested by Delft Geotechnics, they also did back-analyses with the actual failure pressure in a similar manner to Staheli et al. (2010). From their back-analyses, Rostami (2017) obtained an average $R_{p,max}$ of 0.16H. Goerz et al. (2019) did not carry out an average $R_{p,max}$; however, they presented the back-calculated $R_{p,max}$ in a plot as presented in Figure 2.1. Both Rostami (2017) and Goerz et al. (2019) attempted to find a correlation between $R_{p,max}$ and failure pressure; however, there could not find any significant relationship between them.

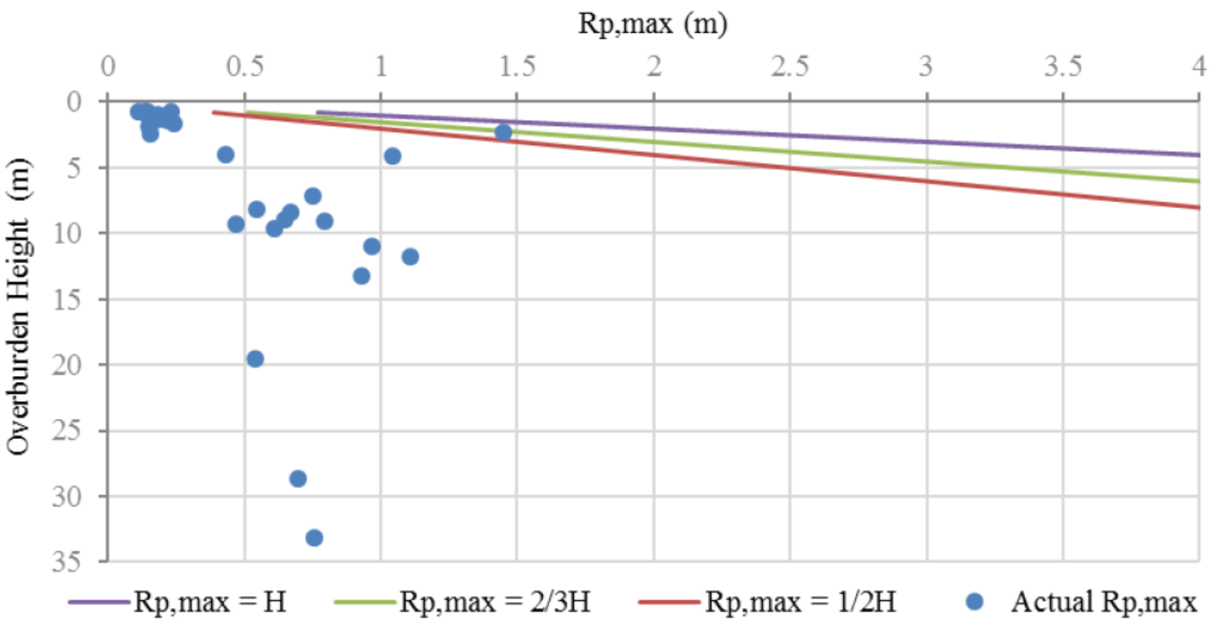

Figure 2.1. Comparison Showing Depth of Cover and Back-Calculated Maximum Plastic Radius (Goerz et al., 2019)

2.2.6. Failure Defined by the Maximum Allowable Strain

While researchers were focusing on the search for a true $R_{p,max}$ based on the pressure measurements from actual hydrofracture failures, another perspective on the failure mechanism of an internally pressurized underground cavity gained popularity. Unlike the past IR assessments, with the Delft method only considered shear failure in the radial direction, a number of researchers started looking into the possibility of tensile failure in the tangential direction. During the HDD construction, as drilling fluid pressure increases, underground cavity deforms in both radial and tangential directions. As Keulen (2001) explained in her thesis, excessive tangential strain is a great threat to the integrity of borehole because it allows pressurized drilling fluid to enter the gap between soil grains on the borehole wall and pushes them even further apart. Such gap can form a kind of wedge, which might initiate crack openings and accelerate hydrofracture failure. Geotechnical materials the surrounding underground cavity, such as soils and rocks, are generally known to be brittle. These brittle materials are typically much weaker in tension, compared to shear and compression; therefore, it is highly probable for them to have large tangential deformation during the cavity expansion, leading to tensile failure. As a result, the underground cavity is likely to have tensile failure by rupturing in tangential direction, rather than having a block failure by shearing in radial direction.

Yu and Houlsby (1991), Verruijt (1993), and Nederlands Normalisatie-Instituut (NEN, 2006) developed cavity expansion models based on the large strain theory, and their models have been applied for IR assessment in HDD. The models of Yu and Houlsby (1991) and Verruijt (1993) are analytical solutions for dilative cohesionless soils, which are based on the assumptions with elastoplastic soil behavior and Mohr-Coulomb yield criterion. These two models allow users to select the tangential strain level and predict the corresponding P_{max}. Unlike the simplicity of Verruijt's (1993) model, Yu and Houlsby's (1991) model is relatively long and complicated. The series expansion term of Yu and Houlsby's (1991) model particularly brings more challenges to practitioners, who attempt to code this model into software. To search the tangential strain that corresponds to actual failure pressure, Keulen (2001) collected five case studies from BTL. Keulen (2001) made a comparison between P_{max} and P_{lim} from the original Delft method, P_{max} from Verruijt (1993)'s model with 2% and 5% tangential strains, and pressure measurements from the actual hydrofracturing failures. From this comparison, it was found that 2% strain is too conservative, and 5% strain is a reasonable tangential strain. Rostami (2017) also attempted to search for the tangential strain that corresponds to the actual failure pressure, and performed back-analysis with the models of Yu and Houlsby (1991) and Verruijt (1993). From the back-calculations, Rostami (2017) obtained tangential strains of 3.3% from Yu and Houlsby's (1991) model, and 4.7% from Verruijt's (1993) model. Another method that applied similar tangential strain value into the existing Delft equation for P_{max} prediction was developed by Nederlands Normalisatie-Instituut (NEN). Since NEN is the Royal Netherlands Standardization Institute, the method developed by NEN has been a standard for the pipeline industry in Netherlands over the years. This NEN method related to IR assessment was introduced in NEN 3650, and for sands, the maximum allowable strain, $\varepsilon_{g,max}$ was defined as 0.05, which is equivalent to 5% (NEN, 2006). Besides the definition of $R_{p,max}$, another major difference between the original Delft method and NEN method is with the application of partial factors to input geotechnical parameters. The NEN method applies partial factors, which were derived based on the field research and laboratory tests presented in NEN 6740. By applying those partial factors, it can mitigate the impact of uncertainty in geotechnical parameter selection. A detailed explanation about the concept and derivation of the partial factors were explained by Guijt et al. (2004). Unlike the models of Yu and Houlsby (1991) and Verruijt (1993), which included dilative behavior of cohesionless soils, the NEN method did not take such plastic behavior into consideration.

2.2.7. Currently Available Modified Versions of Delft Methods Approved by USACE

As explained in previous sections, many researchers have attempted to improve the P_{max} prediction for IR assessment over the years. As a result of all those efforts, they have developed multiple P_{max} prediction models, and practitioners currently have options to choose the most appropriate model for their projects. Currently, the original Delft equation developed by Luger and Hergarden (1988) with $R_{p,max}$ suggested by Delft Geotechnics (van Brussel and Hergarden, 1997) is the most commonly used method for IR assessment. Through experience with past hydrofracture failure occurrences, researchers and practitioners have become aware of the overprediction tendency of Delft method, and they often apply a safety factor of 2 for the calculated P_{max}. The suggestion of Staheli et al (2010) of limiting $R_{p,max}$ to less than 2 to 3 times the bore diameter is another approach to prevent the overprediction of P_{max}, and the USACE Risk Management Center (RMC) has supported this idea during a recent discussion. Miller and Robinson (2019) suggested the application of an additional safety factor of 1.5 for P_{max} predicted with $R_{p,max}$ suggestion of Staheli et al (2010).

2.3. Site Investigation

2.3.1. Standard Penetration Test (SPT)

SPT is a simple and economical in-situ soil testing/sampling technique, which has been used throughout the world over a hundred years. During SPT, a 51 mm diameter open-ended "Split Spoon Sampler" is driven into soil by a 63.5 kg hammer free-falling from a height of 760 mm height (Canadian Geotechnical Society, 2007). The Number of hammer blows required for 300 mm advancement of split spoon sampler is termed as "N-value", and it is often used as an indicator of in-situ soil characteristics. For example, stiffer soil has greater resistance against the penetration of split spoon sampler; thus, the corresponding N-value is likely to be greater than the one obtained from softer soil. Numerous researchers (Gibbs and Holtz, 1956; Meyerhof, 1956; Terzaghi and Peck, 1967; de Mello, 1971; Mayne and Frost, 1989; and others) studied relationships between the N-value and geotechnical parameters from laboratory/field experiments, and they developed correlation models based on their findings. Publications of these correlation works greatly benefited on determination of geotechnical parameters using SPT results; moreover, some geotechnical/foundation design procedures (Meyerhof, 1976; O'Neill and Reese, 1999; Abu-Hejleh et al., 2003; and others) were proposed on the basis of N-values.

2.3.1.1 Standardization of N-value (N_{60})

As mentioned previously, SPT has been preferred for site investigations throughout the world for its simplicity, cost-effectiveness, and wide availability. Despite such advantages, SPT was reported to have poor reproducibility and great variability (Tavenas, 1971), due to the nature of resulting N-values being affected by multiple factors: equipment type, rod length, borehole diameter, etc. Although the SPT stands for "Standard" Penetration Test, equipment used for this method have not been strictly standardized, therefore, many different types of hammers and release mechanisms have been used for SPTs over the world. To demonstrate the effect of equipment variation on SPT result, Kovacs et al. (1981) conducted energy measurements with multiple drilling rigs; moreover, they evaluated the efficiencies of SPT hammers. From their measurements and evaluations, it was found the energy transmitted to split spoon samplers varied from 40 to 90 percent of the free fall energy of SPT hammers, which demonstrated significant variation in hammer efficiencies existing between the drilling equipment and rig setups. Seed et al. (1985) considered adopting safety hammer as a standard, since it was the most commonly used hammer in the US at that time. From collection of test data associated with safety hammer, average hammer efficiency was found to be approximately 60 percent. This average 60 percent was adopted for the adjustments of hammer efficiencies of the other hammers, in order to convert the uncorrected N-value into standardized N-value, N_{60}. Skempton (1986) improved the concept of N_{60} correction of Seed et al. (1985) by including the effects of rod length, sampler type, and borehole diameter, as it is presented in Equation [2.2].

$$N_{60} = \frac{E_m C_B C_S C_R N}{0.60} \quad [2.2]$$

where E_m is hammer efficiency, C_B is borehole diameter factor, C_S is sampling method factor, C_R is rod length factor, and N is N-value.

Clayton (1990) collected various hammer efficiency data from multiple literatures (Seed et al., 1985; Riggs, 1986; Skepmton, 1986; and Décourt, 1989) and summarized them as presented in Table 2.1. To obtain the value of E_m for Equation [2.2], the summary in Table 2.1 can be useful.

Table 2.1. Variation of Rod Energy Ratios for SPT Hammers (Clayton, 1990)

Country	**Hammer Type**	**Hammer Release Mechanism**	**Hammer Efficiency, E_m**
Argentina	Donut	Cathead	0.45
Brazil	Pin Weight	Hand Dropped	0.72
China	Automatic	Trip	0.60
	Donut	Hand Dropped	0.55
	Donut	Cathead	0.50
Columbia	Donut	Cathead	0.50
Japan	Donut	Tombi Trigger	0.78-0.85
	Donut	Cathead 2 turns + special release	0.65-0.67
UK	Automatic	Trip	0.73
USA	Safety	2 turns on cathead	0.55-0.60
	Donut	2 turns on cathead	0.45
Venezuela	Donut	Cathead	0.43

Skempton (1986) referred to the studies related to the effects of rod length (Schmertmann and Palacios, 1979), sampler type (Seed et al., 1985), and borehole diameter (Lake, 1974; and Sanglerat and Sanglerat, 1982), and summarized them as presented in Table 2.2 to provide the correction factors of Equation [2.2]: C_B, C_S, and C_R.

Table 2.2. Approximate Corrections to Measured N-values (Skempton, 1986)

Factor	Equipment Variables	Value
Borehole diameter factor, C_B	65-115 mm (2.5-4.5 in)	1.00
	150 mm (6 in)	1.05
	200 mm (8 in)	1.15
Sampling method factor, C_S	Standard Sampler	1.00
	Sampler without liner (not recommended)	1.20
Rod length factor, C_R	3-4 m (10-13 ft)	0.75
	4-6 m (13-20 ft)	0.85
	6-10 m (20-30 ft)	0.95
	>10 m (>30 ft)	1.00

2.3.1.2 Internal Friction Angle, φ

Meyerhof (1956) and Peck et al. (1974) introduced correlations between the internal friction angle, φ, and N-value with qualitative descriptions of relative density of sands as presented in Table 2.3. Since the concept of standardizing N-value was not established at this time, the correlation models in Table 2.3 were not developed on the basis of N_{60}.

Table 2.3. N vs. φ Relationships of Sands (Meyerhof, 1956, and Peck et al., 1974)

N-Value (blows/ft or 305 mm)	Relative Density	Approximate (degrees) φ	
		Peck et al. (1974)	Meyerhof (1956)
0 to 4	Very Loose	< 28	< 30
4 to 10	Loose	28 to 30	30 to 35
10 to 30	Medium	30 to 36	35 to 40
30 to 50	Dense	36 to 41	40 to 45
> 50	Very Dense	> 41	> 45

In saturated or very fine silty sands, it is probable to have dynamic pore pressure development during SPT, which may result inaccurate measurement of N-value. Especially for the cases with higher N-values, the effect of pore pressure development may become even greater. In order to estimate φ for such soils, it is suggested to make corrections for the measured N-values prior to substituting them into the correlation models of Table 2.3. N-value of saturated or very fine silty sand, which is greater than 15, should be corrected as $15 + (N - 15) / 2$.

Correlations of Meyerhof (1956) and Peck et al. (1974) are definitely useful estimators of φ for their simplicity; however, it should be understood that these models are limited by not including the effect of overburden stress on SPT, which were emphasized by numerous researchers (Seed et al., 1983; Liao and Whitman, 1986; Skempton, 1986; Olsen, 1997; Mayne and Kemper, 1988; and others). De Mello (1971) found a correlation between the φ, N-value, and overburden stress, σ, using the laboratory test data (24 samples of fine sand and 16 samples of coarse sand) obtained by Gibbs and Holtz (1956) of the United States Bureau of Reclamation (USBR). Statistical formulation of de Mello (1971) was based on the Prandtl-Caquot-Buisman idealized theory, and he proved the regressions of all three cases: fine sand, coarse sand, and jointly both sands, are highly significant, by conducting the F-tests to 95 percent confidence limits. Based on the result of statistical regression, de Mello (1971) expressed N-value in terms of φ and σ, as presented in Equation [2.3].

$$N = 4.0 + 0.015\left\{\frac{2.4}{\tan\varphi}\left[\tan^2\left(\frac{\pi}{4}+\frac{\varphi}{2}\right)e^{\pi\tan\varphi}-1\right]+\sigma\tan^2\left(\frac{\pi}{4}+\frac{\varphi}{2}\right)e^{\pi\tan\varphi}\right\} \pm 8.7 \quad [2.3]$$

From the attempt to validate his correlation model with published/unpublished data collection (Kerisel, 1964; Vesic, 1967; Tavenas et al., 1970; Promon S.A.'s São Paulo Metrô project files; and de Mello's own files), de Mello (1971) found reasonable agreement with slight conservatism for the most cases, except the ones at shallower depths. Such poor validity at the shallower depths could have been resulted by the limitation of SPT for measurement of soil resistance at a low level of confinement. When SPT is conducted, energy generated from impact of hammer drop transfers through drilling rods and reaches the contact point between the tip of split spoon sampler and soil. This transferred energy induces internal stress within soil; subsequently, the induced stress causes deformation and displacement of soil structure. For soil at a sufficient depth, the overburden pressure from surcharging soil is high enough to provide a large degree of confinement, which is capable of restricting the deformation and displacement caused during the SPT. While the deformation and displacement of soil structure is controlled, the friction between the soil grains is mobilized without having a significant loss of energy. For this case, the correlation model of de Mello (1971) is capable of estimating an accurate φ of the corresponding soil. In contrast, for soil at a relatively shallow depth, the overburden pressure from surcharging soil would be significantly lower than the one at a greater depth; therefore, confinement might not be sufficient to restrict the deformation and displacement of soil structure caused during the SPT. Without having deformation and displacement of soil structure controlled, a great portion of transferred energy at the contact point between the tip of split spoon sampler and soil will be dissipated; thus, friction can only be mobilized with the remainder energy, which would be much less that the one at a greater depth. Since it is difficult to measure or quantify the amount of energy loss during the ordinary SPT operation, estimation of φ with the correlation of de Mello (1971) for shallow depth may not be accurate. Throughout the validation work for his correlation model, de Mello (1971) concluded with an emphasis on possibility of unacceptable variation and error within the SPT for sands at "very shallow depths". During the ASCE Specialty Conference on In-Situ Measurement of Soil Properties, Schmertmann (1975) interpreted this "very shallow depths" as "less than 2m". For practical application of correlation of de Mello (1971), Décourt (1989) suggested including the effect of aging of soil on the penetration resistance. Since the correlation of de Mello (1971) was developed based on the data of freshly deposited sands at laboratory (Gibbs and Holtz, 1956), the estimation of φ for in-situ normally consolidated sands was considered to be different, due to the existence of particle bonds created by aging. With the N_{60} normalized to the vertical effective stress of 98.1kPa as presented in Equation [2.4], Décourt (1989) modified the correlation of de Mello (1971) as presented in Equation [2.5].

$$(N_1)_{60} = N_{60}\left[\frac{(\sigma'_{oct})_1}{\sigma'_{oct}}\right]^{0.5} \qquad [2.4]$$

where: $(\sigma'_{oct})_1$ is octahedral stress of a normally consolidated sand (98.1 kPa), and σ'_{oct} is octahedral stress corresponding to the depth where SPT is performed (98.1 kPa for derivation of Equation [2.5]).

$$(N_1)_{60} = -5.666 + 0.031\left\{\frac{6.8}{tan\,\varphi}\left[tan^2\left(\frac{\pi}{4}+\frac{\varphi}{2}\right)e^{\pi\,tan\,\varphi}-1\right]+10\,tan^2\left(\frac{\pi}{4}+\frac{\varphi}{2}\right)e^{\pi\,tan\,\varphi}\right\} \qquad [2.5]$$

Kulhawy and Mayne (1990) derived a simplified version of the correlation of de Mello (1971), as presented in Equation [2.6].

$$\bar{\varphi}_{tc} \approx tan^{-1}\left[\frac{N}{\left(12.2+20.3\frac{\bar{\sigma}_{v0}}{P_a}\right)}\right]^{0.34} \qquad [2.6]$$

where: φ'_{tc} is triaxial compression effective stress friction angle, σ'_{v0} is vertical (or overburden) effective normal stress, and P_a is atmospheric pressure (101.3 kPa).

In Geotechnical Engineering Circular (GEC) No. 5, Sabatini et al. (2002) included a modified version of the correlation between the φ' and $(N_1)_{60}$, which was originally found by Hatanaka and Uchida (1996). As it was briefly introduced in Equation [2.4], $(N_1)_{60}$ is the N_{60}, which is normalized to an atmospheric pressure of one. Detailed explanations of the inputs are presented with Equation [2.7].

$$(N_1)_{60} = N_{60}\left(\frac{P_a}{\sigma'_{v0}}\right)^n \qquad [2.7]$$

where: N_{60} is N-value corrected to 60 percent hammer efficiency, P_a is atmospheric pressure (P_a = 98 kPa from Liao and Whitman, 1986; P_a = 100 kPa from GEC No. 5, 2002), σ'_{v0} is vertical effective stress, and n is 0.5 to 0.6 in sands (Seed et al., 1983; Liao and Whitman, 1986; Olsen, 1997), and 1 in clays (Mayne and Kemper, 1988; Olsen, 1997).

For development of correlation model between the φ' and N-value, Hatanaka and Uchida (1996) collected data from the SPT and drained triaxial compression test with high-quality undisturbed sand samples from six different sites (12 samples from fill and naturally deposited sands), which were recovered by in-situ freezing sampling technique. Moreover, Hatanaka and Uchida (1996) also collected data from the other investigators (Uchida et al., 1990; Iai and Kurata, 1991) with reconstituted samples made from Toyoura sand. The type of SPT hammer used for the study by Hatanaka and Uchida (1996) was the "Tonbi" hammer with 78 percent energy efficiency (Yoshimi, 1994); therefore, N-value of this test data was expressed as N_{78}. Drained internal friction angle, φ', can be expressed in terms of normalized N_{78}, $(N_1)_{78}$, as presented in Equation [2.8]. Almost all of the data of Hatanaka and Uchida (1996), fell within the range of ±3°.

$$\varphi' = \sqrt{20(N_1)_{78}} + 20° \qquad [2.8]$$

For practical application of correlation of Hatanaka and Uchida (1996), $(N_1)_{78}$ associated with Japanese Tonbi hammer was required to be converted into $(N_1)_{60}$ of US safety hammer with 60 percent energy efficiency. Sabatini et al. (2002) introduced the converted correlation between the φ' and $(N_1)_{60}$, in GEC No. 5 as presented in Equation [2.9].

$$\varphi' = \sqrt{15.4(N_1)_{60}} + 20° \qquad [2.9]$$

Kulhawy and Chen (2007) questioned about the validity of using sand correlations for the predictions of φ' of very coarse-grained soils. Throughout the comprehensive examinations of shear strength properties on very coarse-grained soils (36 Sands with D_{50} = 0.1-0.3 mm, and D_{50} = 0.1-0.5 mm; 21 Gravels with D_{50} = 1-7 mm, and D_{50} > 7 mm), Chen (2004) was able to compile a database of in-situ shear strength of both sands and gravels. Based on the database of Chen (2007), Kulhawy and Chen (2007) found a correlation between the φ' and $(N_1)_{60}$ as presented in Equation [2.10]. Since the r^2 of this correlation is 0.356, it should be understood that a great scatter exists within the model, which may lower the accuracy of prediction of φ'. Brown et al. (2010) suggested using the correlation of Kulhawy and Chen (2007) as a first-order estimator of φ' for a wide range of cohesionless soils with N-values up to 100.

$$\varphi' = 27.5 + 9.2 \log[(N_1)_{60}] \quad [2.10]$$

In addition, Barthélemy (1974) reported a variation in N-value between the sand samples with different mica content, even though the values of φ obtained from the triaxial test were identical. Mica content difference was only 10%; however, 55% reduction of N-value was observed. Therefore, in order to achieve more accurate estimations of φ of cohesionless soils, it is essential to keep the record of detailed soil descriptions from field and laboratory.

2.3.1.3 Shear Modulus, G

According to the theory of liner elasticity, shear modulus, G, can be defined as presented in Equation [2.11].

$$G = \frac{E}{2(1+\nu)} \quad [2.11]$$

where: E is Young's modulus of elasticity, and ν is Poisson's ratio.

Typical values of Poisson's ratio, ν, of coarse-grained soils and fine-grained soils, which were provided by multiple researchers (Bowles, 1996; Newcomb and Birgisson 1999; and Das and Sobhan, 2013) are presented in Table 2.4 and Table 2.5. There were some other type of empirical correlation models of ν available in literatures (Kulhawy, 1969; Kulhawy et al., 1969; Wroth, 1975; Poulos, 1978; and Trautman and Kulhawy, 1987), which require another geotechnical parameters as inputs such as: Initial void ratio, e_0, Relative density, D_R, Age of soil, t, Overconsolidation ratio, OCR, Diameter of the soil particle that 50% of sample mass is smaller, D_{50}, and Friction angle, φ.

For determination of E of coarse-grained soils, various correlations between the E and N-value of cohesionless soils were collected from multiple literature sources (Ohya et al., 1982; Briaud et al., 1985; Mayne and Frost, 1989; Kulhawy and Mayne, 1990; Yagiz et al., 2008; Bozbey and Togrol, 2010; Kenmogne et al., 2011; Cheshomi and Ghodrati, 2015; Anwar, 2018) as presented in Table 3.6. Unlike most of the correlations of Table 2.6 developed for the estimation of E_{PMT} (equally treated as E), the correlation of Mayne and Frost (1989) results in the DMT modulus, E_D. With a setting of two elastic half spaces in contact with a thin flat circular expandable steel membrane of dilatometer, Marchetti (1980) described the movement of the steel membrane, s_0 as presented in Equation [2.12].

$$s_0 = \frac{2D\Delta p}{\pi} \times \frac{(1-\nu^2)}{E} \qquad [2.12]$$

where: D is the membrane diameter, and Δp is pressure applied from the membrane toward the soil media, which causes s_0 of deflection. By rearranging Equation [2.12], the dilatometer modulus, E_D, can be defined in terms of E and ν as presented in Equation [2.13].

$$E_D = \frac{2D\Delta p}{\pi s_0} = \frac{E}{1-\nu^2} \qquad [2.13]$$

Table 2.4. Typical Values of Poisson's Ratio of Sands with Various Relative Densities (Bowles, 1996; Newcomb and Birgisson 1999; and Das and Sobhan, 2013)

Relative Density	Poisson's ratio, *v*		
	Bowles (1996)	**Newcomb and Birgisson (1999)**	**Das and Sobhan (2013)**
Very Loose	N/A	N/A	N/A
Loose	0.20-0.35	0.20-0.40	0.20-0.40
Medium	0.30-0.40	N/A	0.25-0.40
Dense	0.30-0.40	0.30-0.45	0.30-0.45
Very Dense	N/A	N/A	N/A

Table 2.5. Typical Values of Poisson's Ratio of Various Fine-Grained Soils (Bowles, 1996; Newcomb and Birgisson 1999; and Das and Sobhan, 2013)

Type of Soil	Poisson's ratio, *v*		
	Bowles (1996)	**Newcomb and Birgisson (1999)**	**Das and Sobhan (2013)**
Soft Clays	N/A	N/A	0.15-0.25
Unsaturated Soft Clays	0.10-0.30	N/A	N/A
Saturated Soft Clays	0.40-0.50	0.40-0.50	N/A
Medium Stiff Clays	N/A	N/A	0.20-0.50
Fine-Grained Soils	N/A	0.30-0.50	N/A

Table 2.6. Correlations between the Modulus of Elasticity and N-value of Cohesionless Soils (Ohya et al., 1982; Briaud et al., 1985, Callanan and Kulhawy, 1985; Mayne and Frost, 1989; Yagiz et al., 2008; Bozbey and Togrol, 2010; Kenmogne et al., 2011; Cheshomi and Ghodrati, 2015; and Anwar, 2018)

Author	Year	Test Method	Soil Description	*E* (kPa)
Ohya et al.	1982	PMT	Alluvial and Diluvial sands (From Tokyo, Nagoya, Osaka, and Sakaide)	$E_{PMT} = 9.08P_aN^{0.66}$
Briaud et al.	1985	PMT	Sand and Clay formations (From USA)	$E_{PMT} = 383N$
Callanan & Kulhawy	1985	Not specified	Sands with fines	$E = 5P_aN_{60}$
			Clean NC sands	$E = 10P_aN_{60}$
			Clean OC sands	$E = 15P_aN_{60}$
Mayne & Frost	1989	DMT	Residual sandy silts (From Virginia and Maryland of USA)	$E_D = 22PaN^{0.82}$
Yagiz et al.	2008	PMT	Sandy, Silty, Clayey soils (From Turkey)	$E_{PMT} = 389N_{60} + 4554$
Bozbey & Togrol	2010	PMT	Sandy soils (From Turkey)	$E_{PMT} = 1330{N_{60}}^{0.77}$
Kenmogne et al.	2011	Not Specified	Gravelly sand - Lower bound (From Cameroon)	$E = 2P_aN$
			Gravelly sand - Upper bound (From Cameroon)	$E = 8P_aN$
Cheshomi & Ghodrati	2015	PMT	Silty sand – N-value > 9 (From Mashhad, Iran)	$E_{PMT} = P_a(9.8N_{60} - 94.3)$
Anwar	2018	PMT	• D_{50} < 0.25mm • Fines Content 25 to 50% • Gravel Content < 25% • Max. gravel size ≤ 10mm • Fines are silt or clay with low plasticity	$E_{PMT} = P_a(33.927N^{0.803})$
			• D_{50} 0.25 to 1mm • Fines Content 15 to 30% • Gravel Content 20 to 40% • Max. gravel size ≤ 10mm	$E_{PMT} = P_a(38.428N^{0.7385})$
			• D_{50} > 1mm • Fines Content 0 to 20% • Gravel Content 35 to 70% • Max. gravel size up to 100mm	$E_{PMT} = P_a(178.14N^{0.4398})$

Some researchers (Bowles, 1996; Asperger & Bennett ,2011; and Das and Sobhan, 2013) provided typical values of *E* of various cohesionless and cohesive soils as presented in Table 2.7 and Table 2.8.

Table 2.7. Typical Values of *E* for Various Cohesionless Soils (Bowles, 1996; Asperger and Bennett ,2011; and Das and Sobhan, 2013)

Soil Description		***E* (MPa)**		
Soil Type	**Relative Density**	**Bowles (1996)**	**Asperger & Bennett (2011)**	**Das & Sobhan (2013)**
Loess	N/A	N/A	14-57	N/A
Silt	N/A	2-20	2-19	
Fine Sand	Loose	N/A	8-11	
	Medium		11-19	
	Dense		19-29	
Silty Sand	N/A	5-20	N/A	
Sand	Loose	10-25	10-29	10-28
	Medium	30-50	29-48	N/A
	Dense	50-81	48-77	35-70
Sand and Gravel	Loose	50-150	29-77	N/A
	Medium	N/A	77-96	
	Dense	100-200	96-192	

Table 2.8. Typical Values of *E* for Various Cohesive Soils (Bowles, 1996; Asperger and Bennett ,2011; and Das and Sobhan, 2013)

Soil Description		***E* (MPa)**		
Soil Type	**Description**	**Bowles (1996)**	**Asperger & Bennett (2011)**	**Das & Sobhan (2013)**
Clay	Very Soft	2-15	N/A	N/A
	Soft	5-25	2-4	2-4
	Medium	15-50	4-8	N/A
	Hard	50-100	8-19	6-14
	Sandy	25-250	N/A	N/A
Glacial Till	Loose	10-150	N/A	N/A
	Dense	150-720		
	Very Dense	500-1440		

Another common approach for determining in-situ *G* is using seismic geophysical survey. Compared to the previously mentioned in-situ tests, such as SPT, CPT (Cone Penetration Test), DMT, PMT, etc., are on the high-strain basis, most of the seismic geophysical surveys: seismic reflection test, seismic refraction test, suspension logging test, steady-state vibration (Rayleigh wave) test, spectral analysis of surface wave (SASW) test, seismic cross-hole test, seismic down-hole (up-hole) test, seismic cone penetration test (SCPT), etc., induce shear strain lower than about 3×10^{-4} % (Kramer, 1996). With these seismic geophysical surveys, shear wave velocities can be measured, which are used for computation of the maximum shear modulus, G_{max}. Multiple researchers (Imai and Yoshimura, 1970; Ohba and Toriumi, 1970; Ohta et al., 1972; Ohsaki and Iwasaki, 1973; Imai and Tonouchi, 1982; Seed et al., 1983; Kramer, 1996; and Anbazhagan and Sitharam, 2010) found correlations between the G_{max} and N-value of cohesionless soils as presented in Table 2.9.

Table 2.9. Correlations between the Maximum Shear Modulus and N-value of Cohesionless Soils (Imai and Yoshimura, 1970; Ohba and Toriumi, 1970; Ohta et al., 1972; Ohsaki and Iwasaki, 1973; Imai and Tonouchi, 1982; Seed et al., 1983; Kramer, 1996; Anbazhagan and Sitharam, 2010)

Author	Year	Test Method	Soil Description	G_{max} (kPa)
Imai & Yoshimura	1970	Down-hole test	Peat, Clay, Silt, Sand, Sandy Gravel, Loam	$G_{max} = 9807N^{0.78}$
Ohba & Toriumi	1970	Rayleigh wave test	Sandy, Clayey, Alternate layers (Osaka)	$G_{max} = 11964N^{0.62}$
Ohta et al.	1972	Not specified in detail (Manipulation of data of shear wave velocities)	Tertiary soil, Diluvial sandy and cohesive soil, Alluvial sandy and cohesive soil	$G_{max} = 13631N^{0.72}$
Ohsaki & Iwasaki	1973	Down-hole test	Sandy soils	$G_{max} = 6374N^{0.94}$
Imai & Tonouchi	1982	Not specified in detail (Manipulation of data of shear wave velocities)	All soil types (Alluvial clay and sand, Diluvial clay and sand)	$G_{max} = 14120N^{0.68}$
Seed et al.	1983	Not specified	Not specified	$G_{max} = 6224N$
Kramer	1996	Based on the data of Imai & Tonouchi (1982)	Based on the data of Imai & Tonouchi (1982)	$G_{max} = 15561N^{0.68}$
Anbazhagan & Sitharam	2010	Multichannel analysis of surface waves	Silty sand with less percentage of clay	$G_{max} = 24280N^{0.55}$

Based on the hyperbolic stress-strain relationship, Hardin and Drnevich (1972) defined G in terms of G_{max} as presented in Equation [2.14]; moreover, Equation [2.14] was plotted in Figure 2.2 for graphical presentation of G reduction among the various level of shear strain, γ.

$$G = \frac{G_{max}}{1+\frac{\gamma}{\gamma_r}} \qquad [2.14]$$

where: γ is shear strain, and γ_r is reference shear strain ($\gamma_r = \tau_{max}/G_{max}$).

As it is presented in Figure 2.2, G is significantly reduced as γ increases; therefore, determination of representable G requires range of γ of soil to be known. Typical ranges of γ of cohesionless soils under various confining pressure were suggested by multiple researchers (Weissman and Hart ,1961; Matsushita, Kishida and Kyo, 1967; Seed, 1968;

Donovan, 1968 and 1969; and Silver and Seed, 1969) as presented in Table [2.10]; moreover, reference strain, γ_r, was suggested as lower than about 3×10^{-4} % (Kramer, 1996). By substituting appropriate values of G_{max}, γ, and γ_r, into Equation [2.14], G of soil of interest can be estimated.

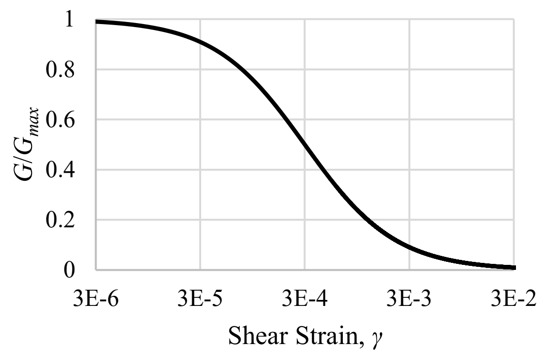

Figure 2.2. Shear Modulus Reduction Curve on the basis of Hyperbolic Stress-Strain Relationship of Hardin and Drnevich (1972)

Table 2.10. Typical Ranges of Shear/Axial Strain of Cohesionless Suggested from Multiple Literatures (Weissman and Hart ,1961; Matsushita, Kishida and Kyo, 1967; Seed, 1968; Donovan, 1968 and 1969; and Silver and Seed, 1969)

Type of Test	**Type of Soil**	**Range of Strain, γ (%)**		**Range of Confining Pressure (kPa)**	**Reference**
		Shear	**Axial**		
Triaxial Compression	Sand and gravel, Silt and sand	N/A	2×10^{-3} to 5×10^{-3}	19 to 86	Weissman and Hart (1961)
	Sand, Silty sand and clayey sand		5×10^{-3} to 0.1	48 to 168	Donovan (1968, 1969)
	Sand		0.1 to 1	144 to 163	Matsushita, Kishida and Kyo (1967)
Simple Shear	Sand	3×10^{-2} to 0.5	N/A	96	Seed (1968)
		10^{-2} to 0.5		24 to 192	Silver and Seed (1969)

2.3.1.4 Undrained Shear Strength, S_u

Despite the complex mechanism of fine-grained soils, many researchers (Sanglerat, 1972; Terzaghi and Peck, 1967; Hara et al. 1974; and others) believed that there is some relationship between the undrained shear strength, S_u, and N-value, and the 36 correlations listed in Table 2.11 are the outcomes of this research. Compared to the classical correlations only having N-value as a variable, recently developed correlations (Sivrikaya and Togrol, 2002; Sivrikaya, 2009; and Nassaji and Kalantari, 2011) also include the plasticity parameters – in-situ water content, *w%*, liquid limit, *LL*, and plastic index, *PI* – as inputs. Moreover, some of the recent correlations (Décourt, 1989; Hettiarachichi and Brown, 1990; Sivrikaya and Togrol, 2002; Sivrikaya, 2009; and Nassaji and Kalantari, 2011) implemented the concept of standardization of N-value (N_{60}) as well.

Table 2.11. Correlations between the Su and N-value of Various Fine-Grained Soils from Multiple Literatures

No.	Author	Year	Soil Description	S_u (kPa)
1	Terzaghi & Peck	1967	Fine-Grained	$S_u = 6.25N$
2	Sanglerat	1972	Clay	$S_u = 12.5N$
3			Silty Clay	$S_u = 10N$
4	Hara et al.	1974	Fine-Grained	$S_u = 29N^{0.72}$
5	Stroud	1974	$PI < 20$	$S_u = 6N$
6			$20 < PI < 30$	$S_u = 4N$
7			$PI > 30$	$S_u = 4.2N$
8	Sowers	1979	Highly Plastic	$S_u = 12.5N$
9			Medium Plastic	$S_u = 7.5N$
10			Low Plastic	$S_u = 3.75N$
11	Nixon	1982	Clay	$S_u = 12N$
12	Ajayi & Balogun	1988	Residual & Lateritic	$S_u = 1.39N + 74.2$
13	Décourt	1989	Clay	$S_u = 12.5N$
14			Clay	$S_u = 15N_{60}$
15	Sivrikaya & Toğrol	2002	CH	$S_u = 4.85N$
16			CH	$S_u = 6.815N_{60}$
17			CL	$S_u = 3.35N$
18			CL	$S_u = 4.925N_{60}$
19			Fine-Grained	$S_u = 4.32N$
20			Fine-Grained	$S_u = 6.18N_{60}$
21			Fine-Grained ($N_{60} < 25$)	$S_u = 0.5(0.9PI + 6.2) \times N_{60}$
22		2006	Fine-Grained	$S_u = 6.09N$
23			CH	$S_u = 7.52N$
24			Clay	$S_u = 6.38N$
25			CL	$S_u = 4.98N$
26			ML	$S_u = 4.22N$
27			MH	$S_u = 3.8N$
28	Sivrikaya	2009	UU	$S_u = 3.33N - 0.75w\% + 0.20LL + 1.67PI$
29			UU	$S_u = 4.43N_{60} - 1.29w\% + 1.06LL + 1.02PI$
30			UCS	$S_u = 2.41N - 0.82w\% + 0.14LL + 1.44PI$
31			UCS	$S_u = 3.24N_{60} - 0.53w\% - 0.43LL + 2.14PI$
32	Hettiarachchi & Brown	2009	Fine-Grained	$S_u = 4.1N_{60}$
33	Nassaji & Kalantari	2011	$PI < 20$ ($r = 0.72$)	$S_u = 1.6N + 15.4$
34			$PI < 20$ ($r = 0.73$)	$S_u = 12.1N_{60} + 17.6$
35			$PI < 20$ ($r = 0.80$)	$S_u = 1.5N - 0.1w\% - 0.9LL + 2.4PI + 21.1$
36			$PI < 20$ ($r = 0.81$)	$S_u = 2N_{60} - 0.4w\% - 1.1LL + 2.4PI + 33.3$

2.3.2. Vane Shear Test (VST)

For fully softened clayey soils with significantly low strength (N-value less than 10), SPT may not be an effective method for measurement of in-situ S_u. In such soils, vane shear test (VST) would be more preferable option. VST can be conducted with the same drilling rig as SPT, and it utilizes vane shear apparatus instead of split spoon sampler. Once the shear vane is driven into the depth of interest with sufficient cover, torque can be applied through the drilling rod until the failure of soil occurs. By using the amount of torque required to reach failure and the dimensions of vane shear apparatus, S_u of the soil can be calculated. If a vane shear apparatus with rectangular blades is used, based on an assumption of the mobilization of uniform shear strength along the blades, $S_{u\ (VST)}$ can be calculated as presented in Equation [2.15].

$$S_{u\,(VST)} = \frac{T}{\pi\left(\frac{d^2 h}{2}+\frac{d^3}{6}\right)} \qquad [2.15]$$

where: T is maximum torque applied to cause failure, d is diameter of the shear vane, and h is height of the shear vane.

However, the value of $S_{u\ (VST)}$ obtained from Equation [2.15] may be different from the actual strength at the failure moment; therefore, a correction should be applied to $S_{u\ (VST)}$ to obtain the true undrained shear strength, S_u. During the review of failure case records of embankments, foundations, strutted excavations, sheet pile walled excavations, and unsupported excavations, Bjerrum (1973) found the values of factor of safety were not in unity; however, they were often greater than 1. When Bjerrum (1973) substituted $S_{u\ (VST)}$ values as the input parameters of stability analysis, he found the values of factor of safety are above 1. However, when Bjerrum (1973) did a back-calculation to obtain S_u, the resulting values were not equivalent to $S_{u\ (VST)}$. Considering the plasticity index effect on the factor of safety, Bjerrum (1973) found a correlation between the plasticity index and correction factor. Multiple researchers (Skempton, 1948; Larsson, 1980; Azzouz et al., 1983; Aas et al., 1986; Chandler, 1988; Mayne and Mitchell, 1988; Mesri, 1989; and Morris and Williams, 1993 and 1994) had similar attempt as Bjerrum (1973), and some of their work is presented in ASTM D2573. Instead of relying on a single method, ASTM committee included multiple literature with different methods, and recommended to have a qualified professional to decide. Since plasticity of cohesive soil is the input for calculation of correction factor, Atterberg limit tests should be conducted in the laboratory to use VST measurements.

REFERENCES

Aas, G., Lacasse, S., Lunne, T., and Hoeg, K., (1986), "Use of In Situ Tests for Foundation Design on Clay," *Use of In Situ Tests in Geotechnical Engineering*, S.P., Clemence, Ed., GSP 6, American Society of Civil Engineers, New York, pp. 1-30.

Abu-Hejleh, N., O'Neill, M.W., Hanneman, D., and Atwoll, W.J., (2003), "Improvement of the Geotechnical Axial Design Methodology for Colorado's Drilled Shafts Socketed in Weak Rock," Report No. CDOT-DTD-R-2003-6, Colorado Department of Transportation, Denver, 192 p.

Ajayi, L.A. and Balogun, L.A. (1988), "Penetration Testing in Tropical Lateritic and Residual Soils – Nigerian Experience," *Proceedings of the First International Symposium on Penetration Testing*, ISOPT-1, Orlando, March 20-24, 1998, Vol. 1, pp. 315-328.

Anbazhagan, P., and Sitharam, T.G., (2010), "Relationship between Low Strain Shear Modulus and Standard Penetration Test 'N' values." *ASTM Geotechnical Testing Journal*, Vol. 33, No. 2, pp. 150-164.

Anwar, M.B., (2018), "Correlation between PMT and SPT Results for Calcareous Soil," *Housing and Building National Research Center Journal*, Vol. 14, No. 1, pp. 50-55.

API, (2009), – *Recommended Practice on the Rheology and Hydraulics of Oil-Well Drilling Fluids*, API RP 13D, Washington, D.C.

Asperger, M., and Bennett, D., (2011), "The Search for G: Selecting Shear Modulus for Use in Hydrofracture Analysis," *Proceedings of 2011 No - Dig Conference*, Washington, D.C., March 27-31, 2011

Azzouz, A., Baligh, M., and Ladd, C.C., (1983), "Corrected Field Vane Strength for Embankment Design," *Journal of Ge6technical Engineering*, Vol. 109, No. 5, pp. 730-734.

Baroid, (1998), – *Baroid Fluids Handbook*, Chapter 9: Rheology and Hydraulics, Baroid Drilling Fluids, Inc., Haliburton, Houston, Texas.

Barthélemy, H.C., (1974), "An Investigation of Relationships between the SPT and the Cone Penetrometer Resistance in Sands," Project Report, Department of Civil Engineering, Duke University.

Bjerrum, L., (1973), "Problems of Soil Mechanics and Construction on Soft Clays and Structurally Unstable Soils (Collapsible, Expansive and Others)," *Proceedings of the 8th International Conference on Soil Mechanics and Foundation Engineering*, Vol. 3, Moscow, pp. 111-159.

Bowles, J.E., (1996), – *Foundation Analysis and Design*, The McGraw Hill Companies, Inc., Fifth Edition, New York

Bozbey, I., and Togrol, E., (2010), "Correlation of Standard Penetration Test and Pressuremeter Data: A Case Study from Istanbul, Turkey." *Bulletin of Engineering Geology and the Enbrivironment*, Vol. 69, pp. 505–515.

Briaud, J.L., Noubani, A., Kilgore, A., and Tucker, L.M., (1985), "Correlation between Pressuremeter Data and Other Parameters," Research Report, Civil Engineering, Texas A&M University, (Cited from Briaud, J.L., The Pressuremeter, Balkema, A.A., Rotterdam, 1992).

Brown, D.A., Turner, J.P., and Castelli, R.J., (2010), – *Drilled Shafts: Construction Procedures and LRFD Design Methods, NHI Course No. 132014 Geotechnical Engineering Circular NO. 10*, Federal Highway Administration, Report No. FHWA NHI-10-016, Washington, DC.

Callanan, J.F., and Kulhawy, F.H., (1985), – Evaluation of Procedures for Predicting Foundation Uplift Movements‖, Report EL-4107, Electric Power Research Institute, Palo Alto, Aug. 1985, 124p.

Canadian Geotechnical Society, (2007), – *Canadian Foundation Engineering Manual*, Canadian Geotechnical Society, Fourth Edition, Richmond, British Columbia, Canada

Chandler, R.J., (1988), "The In-Situ Measurement of the Undrained Shear Strength of Clays using the Field Vane," *Vane Shear Strength Testing in Soils: Field and Lab Studies*, STP 1014, ASTM, West Conshohocken, PA, pp. 13-44.

Chen, J.-R., (2004), "Axial Behavior of Drilled Shafts in Gravelly Soils," Ph.D. Dissertation, Cornell University, Ithaca, N.Y.

Cheshomi, A., and Ghodrati, M., (2015), "Estimating Menard Pressuremeter Modulus and Limit Pressure from SPT in Silty Sand and Silty Clay Soils. A Case Study in Mashhad, Iran," *Geomechanics and Geoengineering*, Vol. 10, No. 3, pp. 194-202.

Clayton, C.R.I., (1990), "SPT Energy Transmission: Theory, Measurement and Significance," *Ground Engineering*, Vol. 23, No. 10, pp. 35-43.

Das, B.M., and Sobhan, K., (2013), – *Principles of Geotechnical Engineering*, 8th Ed., Cengage Learning, Stamford, 2013, 362p.

Décourt, L., (1989), "The Standard Penetration Test State-of-the-Art Report," *International Conference on Soil Mechanics and Foundation Engineering*, Rio de Janeiro.

De Mello, V.F.B., (1971), "The Standard Penetration Test," *4th Panamerican Conference on Soil Mechanics and Foundation Engineering*, San Juan, Puerto Rico, June 1971, Vol. 1, pp. 1-86.

Donovan, N.C., (1968), Personal Communication.

Donovan, N.C., (1969), – Research Brief - Soil Dynamics Specialty Session, *7th International Conference on Soil Mechanics and Foundation Engineering*, Mexico City.

Elwood, D.E.Y., (2008), “Hydraulic Fracture Experiments in a Frictional Material and Approximation for Maximum Allowable Mud Pressure,” M.Sc. Thesis, Queens University, Kingston, Ontario, Canada, July 2008

Gibbs, H.J., and Holtz, W.G., (1956), “Research on Determining the Density of Sands by Spoon Penetration Testing,” *Proceedings of the 4th International Conference on Soil Mechanics*, London, UK, 1957, Vol. 1, pp. 35-39.

Goerz, S., Boelhouwer, N., and Taylor, J., (2019), “A New Perspective in Hydrofracture Analysis,” *Proceedings of 2019 No-Dig Conference*, Chicago, Illinois, March 17-20, 2019

Guijt, W., Vrouwenvelder, T., Grensnigt, N., and Dijkstra, G., (2004), “Safety Concept in The New Dutch Pipeline Standard NEN 3650”, *Proceedings of the Fourth International Offshore and Polar Engineering Conference*, Toulon, France, May 23-28, 2004

Hara, A., Ohta, T., Niwa, M., Tanaka, S., and Banno, T., (1974), “Shear Modulus and Shear Strength of Cohesive Soils,” *Soils and Foundation*, Vol. 14, No. 3, Sept. 1974, pp. 1-12.

Hardin, B.O., and Drnevich, V.P., (1970), "Shear Modulus and Damping in Soils: I, Measurement and Parameter Effects, II. Design Equations and Curves," Technical Reports UKY 27-70-CE 2 and 3, College of Engineering, University of Kentucky, Lexington, Kentucky, July.

Hatanaka, M., and Uchida, A., (1996), “Empirical Correlation Between Penetration Resistance and Internal Friction Angle of Sandy Soils,” *Soils and Foundations*, Vol. 36, No. 4, pp. 1-9.

Hettiarachchi, H. and Brown, T., (2009), “Use of SPT Blow Counts to Estimate Shear Strength Properties of Soils: Energy Balance Approach,” *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 135, No. 6, pp. 830-834.

Iai, S., and Kurata, E., (1991), “Pore Water Pressures and Ground Motions Measured during the 1987 Chiba-Toho-Oki Earthquake,” Note of the Port and Harbour Research Institute, No. 718, pp. 1-18.

Imai, T., and Tonouchi, K., (1982), “Correlation of N-value with S-wave Velocity and Shear Modulus.” *In Proceedings of the 2nd European Symposium on Penetration Testing*, pp. 57-72.

Imai, T., and Yoshimura, Y., (1970), “Elastic Wave Velocity and Soil Properties in Soft Soil.” *Tsuchito-Kiso*, Vol. 18, No. 1, pp. 17-22.

Kenmogne, E., Martin, J.R., and Geofor, S.A., (2011), “Correlation Studies between SPT and Pressuremeter Tests,” *Proceedings of the 15th African Regional Conference on Soil Mechanics and Geotechnical Engineering*, Mozambique, July 18-21, 2011

Keulen, B., (2001), “Maximum Allowable Pressures During Horizontal Directional Drillings Focused on Sand,” Master thesis, Delft University of Technology, Delft, Netherlands, September 2001

Kerisel, J., (1964), “Deep Foundations Basic Experimental Facts,” *Cimientos Profundos*, Mexico, Vol. 1, p. 5.

Kovacs, W.D., Salomone, L.A., and Yokel, F.Y., (1981), “Energy Measurements in the Standard Penetration Test,” *Building Science Series 135*, National Bureau of Standards, Washington, D.C.

Kramer, S.L., (1996), – *Geotechnical earthquake engineering*, Prentice-Hall Inc., New Jersey

Kulhawy, F.H., (1969), “Finite Element Analysis of the Behavior of Embankments,” Ph.D. Dissertation, University of California, Berkeley, 1969, pp. 40-80.

Kulhawy, F.H. and Chen, J.-R. (2007), “Discussion of ‘Drilled Shaft Side Resistance in Gravelly Soils’ by Kyle M. Rollins, Robert J. Clayton, Rodney C. Mikesell, and Bradford C. Blaise,” *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 133, No. 10, pp. 1325-1328.

Kulhawy, F.H., Duncan, J.M., and Seed, H.B., (1969), “Finite Element Analysis of Stresses and Movements in Embankments during Construction,” Contract Report S-69-8, U.S. Army Engineers Waterways Experiment Station, Vicksburg, Nov. 1969, 169 p.

Kulhawy, F.H., and Mayne, P.W., (1990), – *Manual on Estimating Soil Properties for Foundation Design*, Electric Power Research Institute EL-6800, Project 1493-6, Electric Power Research Institute, Palo Alto, California.

Lake, L.M., (1974), “Discussion on Granular Materials,” *Proceedings of Conference on Settlement of Structures*, Cambridge, 1974, Pentech Press, pp. 663.

Larsson, R., (1980), “Undrained Shear Strength in Stability Calculation of Embankments and Foundations on Clays,” *Canadian Geotechnical Journal*, Vol. 17, No. 4, pp. 591-602.

Liao, S.S.C., and Whitman, R.V., (1986), “Overburden Correction Factors for SPT in Sand,” *Journal of Geotechnical Engineering*, ASCE, Vol. 112, No. 3, pp. 373-337.

Luger, H.J., and Hergarden, H.J.A.M., (1988), “Directional Drilling in Soft Soil: Influence of Mud Pressure,” International Society of Trenchless Technology, *No-Dig Conference*, Washington, D.C., October 16-19, 1988

Marchetti, S., (1980), “In Situ Tests by Flat Dilatometer,” *Journal of Geotechnical Engineering Division*, ASCE, Vol. 106, No. GT3, Mar. 1980, pp. 299-321.

Matsushita, K., Kishida, H., and Kyo, K., (1967), "Experiments on Damping of Sands," *Transactions of the Architectural Institute of Japan*, Summaries of Technical Papers (Annual Meeting of AIJ, 1967), p. 166.

Mayne, P.W., and Frost, D.D., (1989), “Dilatometer Experience in Washington, D.C., and Vicinity,” *Transportation Research Record 1169*, Transportation Research Board, Washington, D.C., 1989, pp. 16-23.

Mayne, P.W., and Kemper, J.B., (1988), “Profiling OCR in Stiff Clays by CPT and SPT,” *ASTM Geotechnical Testing Journal*, Vol. 11, No. 2, pp. 139-147.

Mayne, P.W., and Mitchell, J.K., (1988), “Profiling of Overconsolidation Ratio in Clays by Field Vane,” *Canadian Geotechnical Journal*, Vol. 25, No. 1, pp. 150-157.

Mesri, G., (1989), “A Re-evaluation of $S_{u(mob)} = 0.22\sigma'_p$ Using Laboratory Shear Tests,” *Canadian Geotechnical Journal*, Vol. 26, No. 1, pp. 162-164.

Meyerhof, G.G., (1956), “Penetration Tests and Bearing Capacity of Cohesionless Soils,” *Journal of Soil Mechanics and Foundation Division*, ASCE, Vol. 82, No. SM1, Jan. 1956, pp. 1-19.

Meyerhof, G.G., (1976), "Bearing Capacity and Settlement of Pile Foundations," *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 102, No. GT3, March, pp. 197-228.

Miller, M.A., and Robinson, J.L., (2019), “Soil Mechanics and Calculating Hydraulic Fracture Risk,” *Proceedings of 2019 No-Dig Conference*, Chicago, Illinois, March 18-21, 2019

Morris, P.H., and Williams, D.J., (1993), “A New Model of Vane Shear Strength Testing in Soils,” *Geotechnique*, Vol. 43, No. 3, pp. 489-500.

Morris, P.H., and Williams, D.J., (1994), “Effective Stress Vane Strength Correction Factor Correlations,” *Canadian Geotechnical Journal*, Vol. 31, No. 3, pp. 335-342.

Nassaji, F., and Kalantari, B., (2011), "SPT capability to estimate undrained shear strength of fine-grained soils of Tehran, Iran," *Electronic Journal of Geotechnical Engineering*, Vol. 16, 2011, pp. 1229-1238.

NEN, (2006), NEN 3650-1:2003+A1:2006. Eisen voor buisleidingsystemen - Deel 1: Algemene – Katern 1 tot en met 6 (Requirements for pipeline systems - Part 1: General – Quire 1 to 6), in English

Newcomb, D.E., and Birgisson, B., (1999), – NCHRP Synthesis of Highway Practice 278: Measuring In Situ Mechanical Properties of Pavement Subgrade Soils, Transportation Research Board, National Research Council, Washington D.C., 1999.

Nixon, I.K., (1982), "Standard penetration test: state of the art report," *Proceedings of the Second European Symposium on Penetration Testing*, Vol. 1, Amsterdam, 1982, pp. 3-24.

Ohba, S., and Toriumi, I., (1970), "Research on Vibration Characteristics of Soil Deposits in Osaka, Part 2, on Velocities of Wave Propagation and Predominant Periods of Soil Deposits." Abstract, Technical Meeting of Architectural Institute of Japan.

Ohsaki, Y., and Iwasaki, R., (1973), "On Dynamic Shear Moduli and Poisson's Ratio of Soil Deposits." *Soils and Foundations*, Vol. 13, No. 4, pp. 61-73.

Ohta, T., Hara, A., Niwa, M., and Sakano, T., (1972), "Elastic Moduli of Soil Deposits Estimated by N-values." *In Proceedings of the 7th Annual Conference, The Japanese Society of Soil Mechanics and Foundation Engineering*.

Ohya, S., Imai, T., and Matsubara, M., (1982), "Relationships between N-value by SPT and LLT measurement results," *Proceedings of the Second European Symposium on Penetration Testing*, Vol. 1, Amsterdam, 1982, pp. 125–130.

Olsen, R.S., (1997), “Cyclic Liquefaction based on the Cone Penetrometer Test,” *Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils*, National Center for Earthquake Engineering Research, Dec. 1997, State University of New York at Buffalo, pp. 225-276.

O’Neill, M.W., and Reese, L.C., (1999), – *Drilled Shafts: Construction Procedures and Design Methods*, Publication No. FHWA-IF-99-025, Federal Highway Administration, Washington, D.C., p. 758.

Peck, R.B., Hanson, W.E., and Thorburn, T.H., (1974), – *Foundation Engineering*, John Wiley and Sons, Second Edition, New York

,

Poulos, H.G., (1978), “Normalized Deformation Parameters for Kaolin,” Research Report R336, School of Civil Engineering, University of Sydney, pp. 18.

Rostami, A., (2017), “Prediction and Evaluation of Annular Pressure in Horizontal Directional Drilling,” Ph.D. Thesis, University of Alberta, Edmonton, Alberta, Canada, April 2017

Riggs, C.O., (1986), “North American Standard Penetration Test Practice - An Essay,” *Proceedings of ASCE Conference on Use of In-situ Tests in Geotechnical Engineering*, Blacksburg, p. 945-965.

Sabatini, P.J., Bachus, R.C., Mayne, P.W., Schneider, J.A., and Zettler, T.E., (2002), "Evaluation of Soil and Rock Properties," *Geotechnical Engineering Circular No. 5*, Federal Highway Administration, Washington, D.C., p. 385.

Sanglerat, G., (1972), – *The Penetration and Soil Exploration; Interpretation of Penetration Diagrams – Theory and Practice*, Elsevier Publishing Co, Amsterdam

Sanglerat, G., and Sanglerat, T.R.A., (1982), “Pitfalls of the SPT,” *Proceedings of 2nd Europe Symposium on Penetration Testing*, Amsterdam, May 24-27, 1982, Vol. 1, pp. 143-145.

Schmertmann, J.H., (1975), “Measurement of In-Situ Shear Strength,” *Proceedings of ASCE Specialty Conference on In-Situ Measurement of Soil Properties*, Vol. 2, Raleigh, 1975, pp. 57-138. (Closure: pp. 175-179)

Schmertmann, J. H., and Palacios, A., (1979), “Energy Dynamics of SPT,” *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 105, pp. 909-926.

Seed, H. B., (1968), Unpublished Test Results.

Seed, H.B., Idriss, I.M., and Arango, I., (1983), "Evaluation of Liquefaction Potential Using Field Performance Data," *Journal of the Geotechnical Engineering Division*, Vol. 109, No. 3, pp. 458-482.

Seed, H.B., Tokimatsu, K., Harder, L.F. and Chung, R.M., (1985), “Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations,” *ASCE Journal of Geotechnical Engineering*, Vol. 111, No. 12, pp. 1425-1445.

Silver, M.L., and Seed, H.B., (1969), "The Behavior of Sands under Seismic Loading Conditions," Report No. EERC 69-16, University of California, Berkeley.

Sivrikaya, O., and Toğrol, E., (2002), “Relations between SPT-N and q_u,” *5th Intern. Congress on Advances in Civil Engineering*, Istanbul, Turkey, 2002, pp. 943–952.

Sivrikaya, O., and Toğrol, E., (2006), “Determination of undrained strength of fine-grained soils by means of SPT and its application in Turkey”, *Engineering Geology*, Vol. 86, No. 1, 2006, pp. 52-69.

Sivrikaya, O., (2009), “Comparison of Artificial Neural Networks Models with Correlative Works on Undrained Shear Strength”, *Eurasian Soil Science*, Vol. 42, No. 13, 2009, pp. 1487-1496.

Skempton, A.W., (1948), “Vane Tests in the Alluvial Plain of River Forth near Grangemouth,” *Geotechnique*, Vol. 1, No. 2, pp. 111-124.

Skempton, A.W., (1986), "Standard Penetration Test, Procedures and Effects in Sands of Overburden, Relative Density, Particle Size, Aging and Over-consolidation," *Geotechnique*, Vol. 36, No.3, pp. 425-447.

Sowers, G.F., (1979), – *Introductory Soil Mechanics and Foundations: Geotechnical Engineering*, Macmillan, Fourth Edition, New York

Staheli, K., Bennett, D., O'Donnell, H.W., and Hurley, T.J., (1998), "Installation of Pipelines beneath Levees Using Horizontal Directional Drilling," CPAR-GL-98-1, April 1998

Staheli, K., Price, C.G., and Wetter, L., (2010), "Effectiveness of Hydrofracture Prediction for HDD Design," *Proceedings of 2010 No - Dig Conference*, Chicago, Illinois, May 2-7, 2010

Stroud, M.A., (1974), "The Standard Penetration Test in Insensitive Clays and Soft Rocks," *Proceedings of the European Symposium on Penetration Testing*, Stockholm, June 5-7, 1974, Vol. 2, No. 2, pp. 367-375.

Tavenas, F.A., Capelle, J.F., and La Rochelle, P., (1970), "Étude des Sables submergés par échantillonnage non remanié," (Study of Submerged Sands by Undisturbed Sampling), *Canadian Geotechnical Journal*, Vol. 7, No. 1, p. 37.

Tavenas, F.A., (1971), "Discussion to de Mello (1971)," *4th Panamerican Conference on Soil Mechanics and Foundation Engineering*, Puerto Rico, Vol. III, pp. 64-69.

Terzaghi, K. and Peck, R.B., (1967), – *Soil Mechanics in Engineering Practice*, John Wiley and Sons, Second Edition, New York

Trautmann, C.H., and Kulhawy, F.H., (1987), "CUFAD – A Computer Program for Compression and Uplift Foundation Analysis and Design," Report EL-4540-CCM, Vol. 16, Electric Power Research Institute, Palo Alto, Oct. 1987, 148p.

Uchida, A., Kano, H., and Tokimatsu, K., (1990), "Correlation between Dynamic Properties and Static Properties of Sands obtained by In-Situ Freezing," *25th Annual Meeting of JSSMFE*, Vol. 1, pp. 767-768.

USACE, (2007), Hurricane and Storm Damage Reduction System Design Guidelines. Interim., U.S. Army Corps of Engineers, New Orleans District, October 23, 2007

Van Brussel, G.G., and Hergarden, H.J.A.M., (1997), "Research CPAR program: Installation of Pipelines beneath Levees Using Horizontal Directional Drilling," Department of Foundations and Underground Engineering, Delft Geotechnics, Delft, Netherlands, pp. 1-16.

Vesic, A.S., (1967), "A Study of Bearing Capacity of Deep Foundations," Final Report, Georgia Institute of Technology, Atlanta, Georgia

Verruijt, A., (1993), *Grondmechanica*, 4th Ed., Delftse Uitgevers Maatschappij, Delft, Netherlands

Wroth, C.P., (1975), "In-Situ Measurement of Initial Stresses and Deformation Characteristics," *Proceedings of ASCE Specialty Conference on In-Situ Measurement of Soil Properties*, Vol. 2, Raleigh, pp. 180-230.

Xia, H., (2009), "Investigation of Maximum Mud Pressure within Sand and Clay during Horizontal Directional Drilling," Ph.D. Thesis, Queens University, Kingston, Ontario, Canada, January 2009

Yagiz, S., Akyol, E., and Sen, G., (2008), "Relationship between the Standard Penetration Test and the Pressuremeter Test on Sandy Silty Clays: A Case Study from Denizli." *Bulletin of Engineering Geology and the Environment*, Vol. 67, pp. 405-410.

Yoshimi, Y., (1994), "Relationship among Liquefaction Resistance, SPT N-value and Relative Density for Undisturbed Samples of Sands," Tsuchi-to-Kiso, JSSMFE, Ser. No. 435, Vol. 42, No. 4, pp. 63-37.

Yu, H.S., and Houlsby, G.T., (1991), “Finite Cavity Expansion in Dilatant Soils: Loading Analysis,” *Geotechnique*, 41, No. 2, pp. 173-183.

3. Prediction of Annular Pressure Limits using Parameters from Standard Penetration Test

3.1. INTRODUCTION

Delft method used for the estimation of P_{max}, requires multiple geotechnical parameters as its inputs: cohesion, c (S_u for undrained soils), friction angle, φ, and shear modulus, G, as presented in Equation [2.1]. These geotechnical parameters can be obtained by either direct measurements from experimental methods, or approximations based on empirical methods. Measurements of experimental methods can be taken from both laboratory (Direct Shear Test, Triaxial Compression Test, etc.) and field (Pressuremeter Test, Dilatometer Test, Vane Shear Test, etc.). Approximations based on empirical methods are generally made with previously published and widely known correlations found between the in-situ test (Mostly Standard Penetration Test or Cone Penetration Test) results and geotechnical parameters. Since the direct measurements of geotechnical parameters from experimental methods are generally associated with detailed observations of soil behaviors, resulting P_{max} predictions become more accurate and reliable, compared to the ones approximated using empirical methods. Despite such advantages with experimental methods, majority of them are not used for typical HDD projects, because the time and cost required for some experiments can easily exceed the project allowances. Therefore, instead of taking direct measurements from experimental methods, approximating geotechnical parameters based on the empirical methods have been preferred for predictions of P_{max} among the HDD industry. Currently, the most commonly used empirical method for P_{max} prediction of HDD is on the basis of the Standard Penetration Test (SPT), and practitioners have adopted empirical approaches using N-value for determination of input geotechnical parameters of Delft equation: c, S_u, φ, and G, in order to estimate the P_{max}.

Despite the advantages of using Delft method on the basis of SPT results, a tendency of Delft method to overpredict P_{max} has been reported by multiple researchers (Keulen, 2001; Elwood, 2008; Xia, 2009; Staheli et al., 2010; Rostami, 2017; Goerz et al., 2019; etc.) from their laboratory/field hydrofracture records. Most of their comparisons between the Delft P_{max} predictions and hydrofracture pressure measurements did not show good agreements, nor apparent correlations. Such overpredictions and weak correlations could have been caused by imperfect representation of underground cavity of Delft method, or uncertainties underlying within the accuracy of input geotechnical parameters.

According to researchers' (Keulen, 2001; Elwood, 2008; Xia, 2009; Staheli et al., 2010; and Rostami, 2017) explanations on their methodologies, most of the input geotechnical parameters for their P_{max} predictions were not directly measured through experiments; however, they were either estimated on the basis of N-values and soil descriptions from site investigation reports (Details about these estimations were not explained. It is probable that borehole logs with SPT results were provided and the geotechnical parameters were estimated based on the given N-values.), or selected from typical values available in publications. As mentioned previously, similar procedures with SPT have been often used among the HDD industry; therefore, resulting P_{max} predictions could have been inaccurate; furthermore, it might have affected on projects negatively by increasing risks of hydrofracture occurrences.

Determination of accurate geotechnical parameters based on the N-value is often challenging, due to the following reasons: weak correlations between the N-values and geotechnical parameters, difficulty with choosing an appropriate correlation model from numerous options available in literatures, limited material information obtained from SPT. First of all, it is known the N-values and geotechnical parameters (especially c, S_u, and G) of soils typically have poor correlations. Soils do not have simple mechanisms and their behaviors are governed by various factors, such as: composition of material, in-situ water content, pore water pressure, strain level, etc. This means, even though the N-values obtained from two soils are the same, their behaviors and geotechnical parameters can be varied from each other. Secondly, selecting an appropriate correlation model for soil is challenging, especially when there are great number of options available. Since the SPT has been broadly used for site investigations, numerous researchers attempted to make this test more applicable by linking the data between N-value and geotechnical parameters together. By result, many correlation models have been developed, which can be challenging practitioners while they make their selections for the soil models. Lastly, information about the soil characteristics obtained from SPT is limited for the estimation of all geotechnical parameters. By conducting a typical SPT, it is expected to obtain the followings: soil samples, field drilling logs, N-values, and pocket penetrometer measurements (Generally, additional vane shear tests for very soft soils and installation of piezometers for groundwater level monitoring are also accessible upon the request). Once the sampled soils arrive to laboratory, they are used for soil classification by conducting various index tests, such as water content test, gradation test, and plasticity test. Based on these laboratory/field works associated with SPT, borehole logs can be completed and included in geotechnical investigation reports. For determination of geotechnical parameters, these information from geotechnical reports are definitely helpful; however, there are still more important information required to fully understand the soil behaviors, such as: pore water pressure response, stiffness, volume change, and others. Without having sufficient information and understanding about the soils, accuracy of estimations of geotechnical parameters might remain questionable. In order to overcome the uncertainties within determination of geotechnical parameters on the basis of SPT results, practitioners are often required to make their own judgements. Making good judgement requires firm knowledge and experience in site investigation and geotechnical engineering; however, this is not the case for every practitioner. Moreover, while practitioners make their own judgements for determination of input geotechnical parameters, there is possibility of procedures of P_{max} prediction becoming subjective and unsystematic. To avoid subjective decisions and improve the accuracy of P_{max} prediction using Delft method, a more direct and systematic method for determination of input geotechnical parameters was considered.

In Chapter 3, a newly developed SPT-based P_{max} prediction method is introduced. For development of the new SPT-based method, two possible scenarios of underground cavity expansion of HDD bores were hypothesized, considering the effects of soil types and corresponding hydraulic conductivities: drained conditions with coarse-grained soils, and undrained conditions with fine-grained soils. Multiple literatures (Mayne and Frost, 1989; Kulhawy and Mayne, 1990; Terzaghi et al., 1996; and others) related to the correlation models found between the N-values and laboratory/field-based geotechnical parameters: c, S_u, φ, and G, were reviewed and compared, in order to search for the most representable and reasonably conservative ones; moreover, they were incorporated into the new SPT-based method, so that P_{max} can be predicted for a given N-value directly. To conserve the current standard practice of P_{max} prediction in HDD design, the new method was developed on the basis of Delft method. The new SPT-based method establishes a systematic design guideline for P_{max} prediction in HDD by providing suggestions for determination of input geotechnical parameters with appropriate correlation models. By following the systematic procedure of the new SPT-based method, subjective decisions for geotechnical parameters can be avoided; furthermore, accuracy of resulting P_{max} prediction is expected to be improved. Hydrofracture case study data from multiple literatures (Keulen, 2001; Elwood, 2008; Xia, 2009; and Staheli et al., 2010) was collected to test the validity and conservatism of the new SPT-based method. Obtained P_{max} predictions using the new method were slightly less than the predictions made by the authors of the literatures; moreover, they were closer to the actual hydrofracture pressure measurements. For assistance of practitioners with graphical understanding, algorithm of the new SPT-based method is also presented in a flow chart form.

3.2. SPT-BASED METHOD

3.2.1. Standardization of N-value (N_{60})

For the SPT-based method, it is also preferable to have N-value standardized as N_{60}, to avoid any potential error caused by the wide variability of SPT. Detailed procedures of the standardization of N-value is included in the Chapter 2 of this thesis. Values of C_B, C_S, and C_R, those represent the SPT equipment and borehole size, can be selected from Table [2.1], and [2.2], and they can be substituted into Equation [2.2] to obtain N_{60}.

3.2.2. Drained Soil Model

HDD is a process involving drilling mud, which is generally consisted of water, bentonite, and optional additives. Drilling mud forms a thin low permeable layer of filter cake on annulus wall, which provides sealing of pore spaces between soil grains and allows pressure to be contained inside the HDD bore. For a successful HDD operation, sufficient amount of pressure must be applied to the drilling mud, in order to maintain the stability of borehole and continual transportation of cuttings. Once the proper sealing of borehole wall is achieved by the formation of filter cake layer, pressure in drilling mud begins to develop in outward direction; subsequently, it induces internal stress in surrounding soil media. While the internal stress is distributed to each component of soil: solid grains, water, and air (if the soil is in unsaturated condition), behavior and load-resisting mechanism of the soil are governed by the response of water existing in pore spaces. Coarse-grained soils with low fine contents, such as sands and gravels, are typically known to be highly permeable, because their relatively large pore spaces may become an excellent pathway for the flow of pore water. Therefore, once this type of soil is externally loaded compressively, pore water is likely to be dissipated through those pore spaces without having significant pressure built up. As the pore water drains out in response to external loading, soil grains would be brought into closer contacts to each other. If the application of external loading continues, friction between soil grains would be mobilized, which would support the soil mass to resist against the further loading and consequential deformation. Thus, coarse-grained soil with low fine contents is considered to have drained behavior with mobilized friction as the governing load-resisting mechanism against external compressive loading. Since the coarse-grained soil with low fine contents is generally known to be cohesionless, c of Equation [2.1], can be considered to be negligible. By assuming the c as zero, Delft method of Equation [2.1] for drained soil model can be simplified as presented in Equation [3.1].

$$P_{max} = u + [\sigma'_0(1 + sin\,\varphi)] \times \left[\left(\frac{R_0}{R_{p,max}}\right)^2 + \frac{\sigma'_0\, sin\,\varphi}{G}\right]^{\frac{-\,sin\,\varphi}{1+sin\,\varphi}} \quad [3.1]$$

3.2.2.1. Initial Effective Stress, σ'_0

When Luger and Hergarden (1988) introduced Delft method for the first time, they did not clearly define the initial effective stress, σ'_0. In attempt to provide a suggestion for σ'_0, Luger and Hergarden (1988) made a comparison between the P_{max} predictions using both analytical approach (Delft method) and numerical approach (Finite element analysis with PLUTO), and they concluded that horizontal effective stress, σ'_h, and vertical effective stress, σ'_v, may be used as bounds for determination of σ'_0. For the case with large gap between the σ'_h and σ'_v, suggestion of Luger and Hergarden (1988) may result σ'_0 in a wide range; subsequently, it can cause significant error in the prediction of P_{max} as well. By referring to NEN 3651, Keulen (2001) introduced more rigorous definition, which takes an average between the σ'_h and σ'_v as the value of σ'_0. Compared to σ'_v being simply approximated using bore depth and unit weight of surcharge soil in general, estimation of σ'_h requires more complicated procedure, since σ'_h is strongly dependent on geological setting and history of site. In order to measure the σ'_h, it requires pressuremeter test (PMT) or dilatometer test (DMT) to be conducted; however, as previously mentioned, direct measurement with additional in-situ test requires more time and cost, which would not be feasible for every project (Anwar, 2018). Due to such limitation, majority of practitioners (Xia, 2009; Rostami, 2017; Miller and Robinson, 2019; Neher and Bennett, 2019; Andresen and Staheli, 2019; Landing, 2019; and others) preferred considering σ'_v as σ'_0 for simple application of Delft method. For development of the SPT-based P_{max} prediction method of Chapter 3, it was decided to conserve the preference of HDD industry; therefore, the σ'_0 was defined as σ'_v as well.

3.2.2.2. Maximum Allowable Radius of the Plastic Zone, $R_{p,max}$

In order to predict the P_{max} using Delft method, it is required to estimate the size of plastic zone around HDD borehole. When Luger and Hergarden (1988) introduced Delft method for the first time, they suggested the maximum allowable radius of plastic zone, $R_{p,max}$, to be less than the depth of soil cover. By following this suggestion with $R_{p,max}$, it is expected to have the plastic zone reaching all the way up to the ground surface; furthermore, plastic behavior of surficial soil may result crack openings and lead the HDD bore into hydrofracture failure. During the Construction Productivity Advancement Research (CPAR) program, which was conducted for the development of guidelines: "Installing Pipelines beneath Levees using Horizontal Directional Drilling", USACE (Staheli et al., 1998) required a safety measure to lower the risk of hydrofracture. To support the CPAR with a conservative prediction method for P_{max}, van Brussel and Hergarden (1997) prepared the "Delft Geotechnics Report", suggesting $R_{p,max}$ to be chosen as half of the depth of cover in clayey/peat layers ($R_{p,max} = H/2$), and two third of the depth of cover in sand layer ($R_{p,max} = 2/3\ H$). Since the guidelines for HDD were established by USACE, practitioners have used the $R_{p,max}$ suggestion of van Brussel and Hergarden (1997) extensively. However, despite the adoption of conservative $R_{p,max}$ of van Brussel and Hergarden (1997), the tendency of Delft method to overpredict P_{max} kept occurring. Multiple researchers (NEN, 2006; Staheli et al., 2010; Rostami, 2017; Goerz et al., 2019; and others) have attempted to back-calculate the $R_{p,max}$ using the actual hydrofracture pressure measurements from various case studies; nevertheless, a clear definition of $R_{p,max}$ has not been found up to the present. Since the major scope of Chapter 3 is on development of new P_{max} prediction method on the basis of SPT, controversy over the $R_{p,max}$ definition of Delft method would not be further discussed for brevity. For the new SPT-based method, preference of the $R_{p,max}$ suggestion of van Brussel and Hergarden (1997) among the HDD industry is conserved; moreover, application of an additional safety factor is considered for the mitigation of overprediction of P_{max}, , which would be discussed in further section.

3.2.2.3. Internal Friction Angle, φ

For development of new SPT-based P_{max} prediction method, it was required decide which model would be the most appropriate for estimation of φ of soil, among the previously introduced seven correlations, which are introduced in Chapter 2: Meyerhof (1956), de Mello (1971), Peck et al. (1974), Décourt (1989), Kulhawy and Mayne (1990), Sabatini et al., (2002), and Kulhawy and Chen (2007). In order to compare the seven models at various stress levels, correlations were plotted for σ'_{v0} = 40 kPa, σ'_{v0} = 150 kPa, and σ'_{v0} = 300 kPa, as presented in Figure 3.1, 3.2, and 3.3.

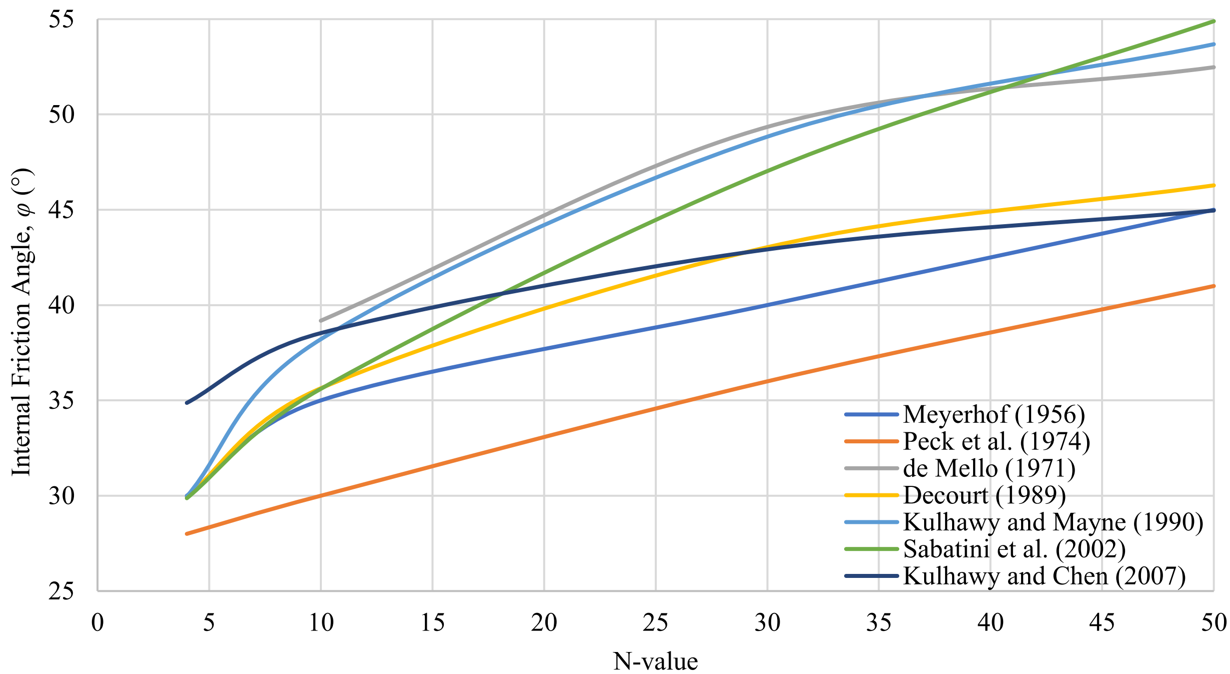

Figure 3.1. Correlations between φ and N-value at σ'_{v0} = 40 kPa

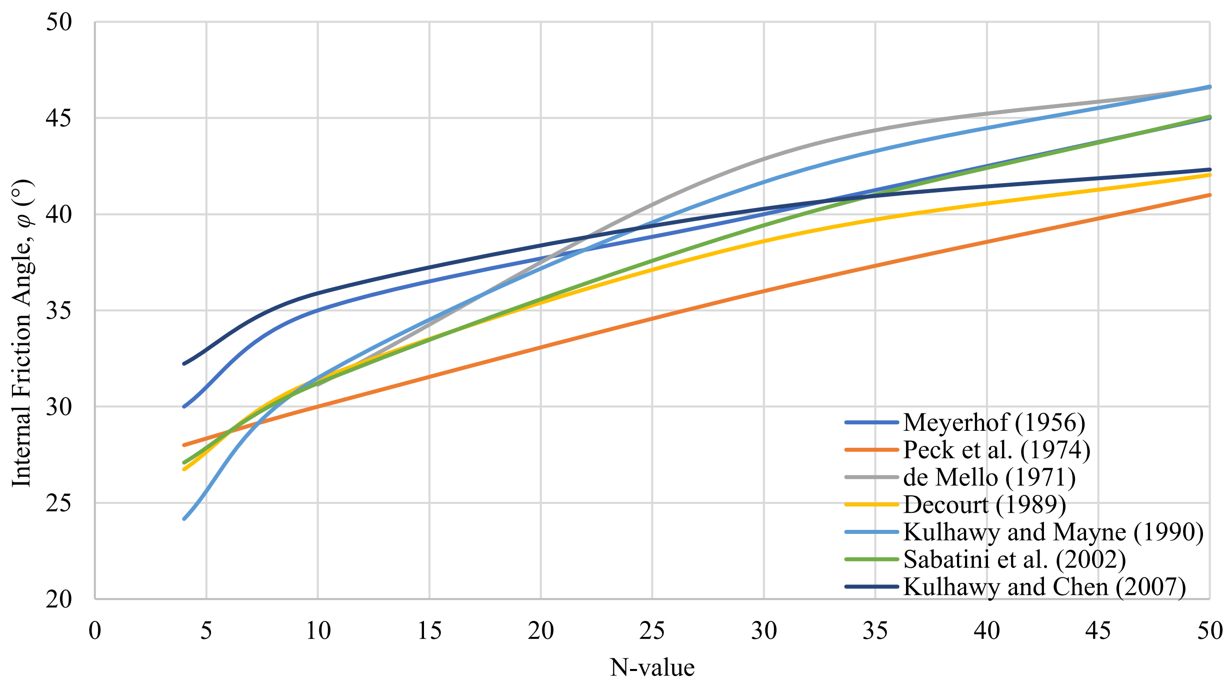

Figure 3.2. Correlations between φ and N-value at σ'_{v0} = 150 kPa

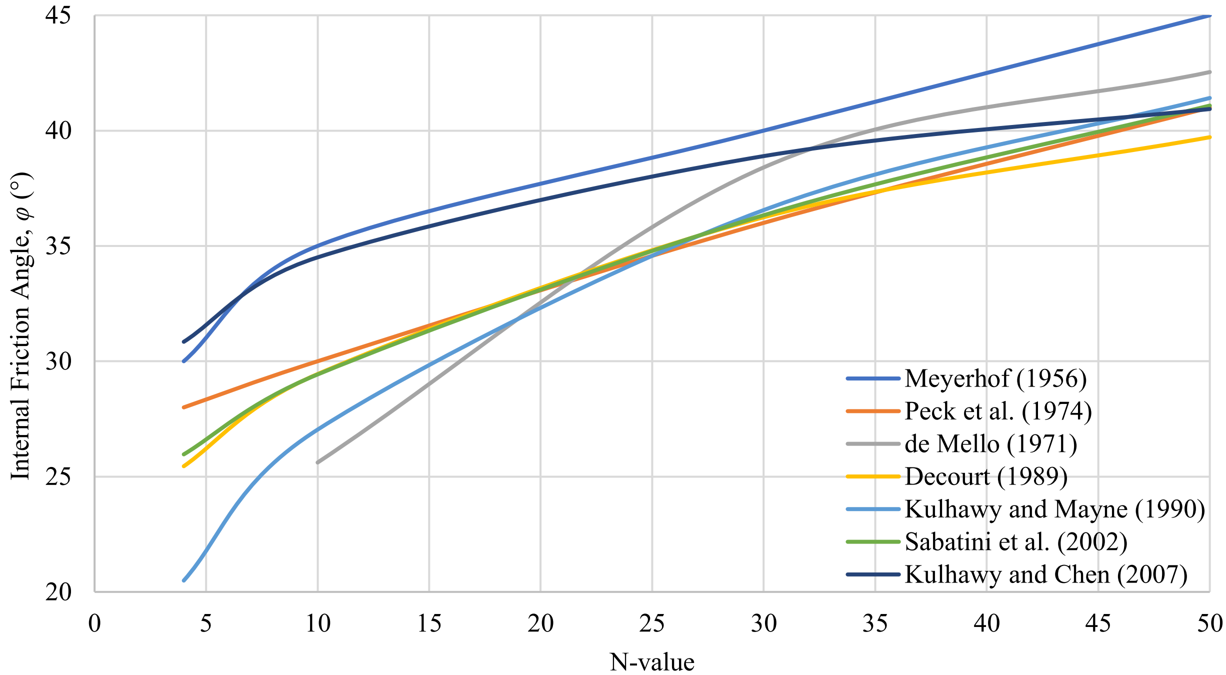

Figure 3.3. Correlations between φ and N-value at σ'_{v0} = 300 kPa

The 40 kPa overburden stress of Figure 3.1 is an approximate setting for depth of cover of 2 m, which is the interpretation of "very shallow depth" provided by Schmertmann (1975) for SPT. As de Mello (1971) mentioned about the possibility of variation and error within the SPT results for sands at "very shallow depth", correlations of Figure 3.1 for shallow depth was found to deviate larger than the ones of Figure 3.2 and 3.3 for deeper depths. Correlation models of Meyerhof (1956) and Peck et al. (1974) are advantageous with their simplicities; however, their predictions of φ do not include the effect of overburden stress. As it can be found from Figure 3.3, Meyerhof's (1956) correlation tends to predict unconservative φ at high level of overburden stress. The correlation of Peck et al. (1974) predicts excessively conservative φ at low to medium stress levels as it is presented in Figure 3.1 and 3.2. Compared to the simple correlations of Meyerhof (1956) and Peck et al. (1974), the ones developed by de Mello (1971) and Décourt (1989) are lengthier and more complicated. Décourt (1989) improved the correlation of de Mello (1971) by including the effect of aging of soil; moreover, he embedded N-value correction that reflects the difference between the efficiencies of SPT hammers used for laboratory and field studies; therefore, Décourt (1989)'s correlation was considered more realistic than the previously developed ones. From Figure 3.1 and 3.2, it can be observed that correlation of de Mello (1971) makes unconservative predictions of φ at low to medium stress levels. The correlation of Décourt (1989) predicts excessively conservative φ, especially at medium to high stress levels. Since Kulhawy and

Mayne (1990) simplified the correlation developed by de Mello (1971), their model also predicts excessively unconservative φ at low to medium stress levels as it can be observed in Figure 3.1 and 3.2. The correlation of Kulhawy and Chen (2007) was developed for very coarse-grained soils and its predictions of φ are close to the mean values of all seven models for all stress levels; however, as it can be observed from Figure 3.1, 3.2, and 3.3, the predictions associated with low N-values are found excessively unconservative. Thus, the correlation of Kulhawy and Chen (2007) was expected to result risky prediction for loose or very loose soils; it was considered to be more applicable for soils, coarser than sands. The correlation of Sabatini et al. (2002) also makes φ predictions close to the mean values of all models at all stress levels; nevertheless, as it can be found in Figure 3.1, predictions with N-values greater than 40 at low stress level tend to be excessively unconservative. Since the range of unconservative prediction is not significantly wide and the modelling of Sabatini et al. (2002) is relatively simple, this correlation is considered to be the most appropriate one for the estimation of φ for newly developed SPT-based P_{max} prediction method. By substituting the definition of $(N_1)_{60}$ of Equation [2.7] into Equation [2.9] with P_a as 100 kPa and n as 0.5, correlation of Sabatini et al. (2002) could be expressed as presented in Equation [3.2].

$$\varphi' = \sqrt{15.4 N_{60} \left(\frac{100\text{kPa}}{\sigma'_{v0}}\right)^{0.5}} + 20° \qquad [3.2]$$

3.2.2.4. Shear Modulus, G

Soil is generally known as a complex material with non-linear deformation behavior, which means the slope of stress-strain (τ vs. γ) curve, shear modulus, G, varies along the different strain levels. Therefore, in order to determine the representable G of such material for an accurate prediction of P_{max}, it is important to know the valid range of shear strain of hydrofracture occurrences. For current design approaches, an assumption of limiting the deformation of HDD bore within elastic range is preferred, because it is difficult to predict the inelastic behavior of largely deformed soil accurately; moreover, a bore with large deformation in tangential direction is subject to have crack openings initiated on annulus wall. Further pressurization of drilling mud may propagate these crack openings and ultimately lead into hydrofracture; hence, practitioners prefer their designs to be limited within the elastic range of soil. According to the derivation of original Delft method (Luger and Hergarden, 1988) of Equation [2.1], G is an elastic parameter of soil around HDD bore. As it is presented in Equation [2.11], determination of elastic G requires modulus of elasticity, E, and Poisson's ratio, v, to be known.

For determination of the v, drainage condition of soil should be identified. Unlike the volume of soil at undrained loading condition remains constant, the one of soil at drained loading condition changes by amount of water dissipated from its pore spaces. With assumptions of constant volume and small strain, theoretical v of undrained soil can be simply calculated as 0.5; however, the one of drained soil cannot be determined in a similar manner, due to the variability of v caused by the volume change. Despite the extensive usages of SPT method in determination of geotechnical parameters, direct correlation between the v and N-value has not been found. There are some other types of empirical correlation models of v available in literatures (Kulhawy, 1969; Kulhawy et al., 1969; Wroth, 1975; Poulos, 1978; and Trautman and Kulhawy, 1987), which require additional geotechnical parameters as inputs, such as the initial void ratio, e_0, relative density, D_R, age of soil, t, overconsolidation ratio, OCR, diameter of the soil particle for which 50% of the sample mass is smaller, D_{50}, and friction angle, φ. These geotechnical parameters can be obtained by either empirical estimations, or direct measurements from additional field/laboratory tests. Since the determination of v using empirical correlation models (Kulhawy, 1969; Kulhawy et al., 1969; Wroth, 1975; Poulos, 1978; and Trautman and Kulhawy, 1987) with empirically estimated input geotechnical parameters is an approximation on the basis of another approximations, it may accumulate the possible errors associated with each correlation and result in the estimation of v with questionable accuracy. Conducting additional field/laboratory tests may allow better estimation of v; however, this approach does not serve the original purpose of making prediction of P_{max} more efficiently with new SPT-based method. Therefore, instead of making an estimation of v with questionable accuracy or spending more resources on additional field/laboratory tests, a simpler approach of adapting typical values of v from literature reports (Bowles, 1996; Newcomb and Birgisson 1999; Das and Sobhan, 2013; and others) was considered. These literature reports did not directly correlate the typical values of v with N-values; however, they provided suggestions for ranges of v with corresponding relative densities of soils, which are described in a qualitative manner. To link the v and N-value directly for development of SPT-based method, the relationship between the N-value and qualitatively described relative density of sands in Table 2.3 (Peck et al., 1974) was adopted. Suggested values of v of Table 2.4 (Bowles, 1996; Newcomb and Birgisson, 1999; and Das and Sobhan, 2013), which showed good agreement with each other, were linked to the relationship between the N-value and qualitatively described relative density of Peck et al. (1974), as they are presented together in Table 3.1. Based on the suggestions of Bowles

(1996), Newcomb and Birgisson (1999), and Das and Sobhan (2013), the writer created a relationship between the v and N-value with a minor modification and rearranged it as presented in Table 3.2. The upper bound N-value of data of Table 3.2 was limited to 100, since typically SPT is done only up to 50 blow counts to prevent damage to the split spoon sampler (the N-value is the total blow counts required for 12 inches advancement of split spoon sampler, and each blow count represents 6 inches advancement and any test result with a blow count higher than 50 is termed as "Refusal"). For development of the correlation model between the v and N-value, the data in Table 3.2 was plotted as presented in Figure 3.4. Two correlation models were created, which are 6th degree polynomial function and cubic function. The 6th degree polynomial function is closer to the data of Table 3.2; however, it should be understood that this data is also based on a mere approximation. Therefore, a simpler cubic function may be considered as another option for the development of SPT-based method.

Table 3.1. Relationships between the Typical Values of Poisson's Ratio and N-values of Sands Associated with Relative Densities (Peck et al., 1974; Bowles, 1996; Newcomb and Birgisson 1999; and Das and Sobhan, 2013)

N-value	**Relative Density (Peck et al., 1974)**	**Poisson's ratio, v**			
		Bowles (1996)	**Newcomb and Birgisson (1999)**	**Das and Sobhan (2013)**	**Modified by Writer**
0-4	Very Loose	N/A	N/A	N/A	0.10-0.20
4-10	Loose	0.20-0.35	0.20-0.40	0.20-0.40	0.20-0.25
10-30	Medium	0.30-0.40	N/A	0.25-0.40	0.25-0.30
30-50	Dense	0.30-0.40	0.30-0.45	0.30-0.45	0.30-0.40
Over 50	Very Dense	N/A	N/A	N/A	0.40-0.45

Table 3.2. Poisson's Ratio and N-value of Cohesionless Soil

N-value	**Poisson's ratio, v**
$N = 0$	0.10
$N = 5$	0.20
$N = 10$	0.25
$N = 30$	0.30
$N = 50$	0.40
$N = 100$	0.45

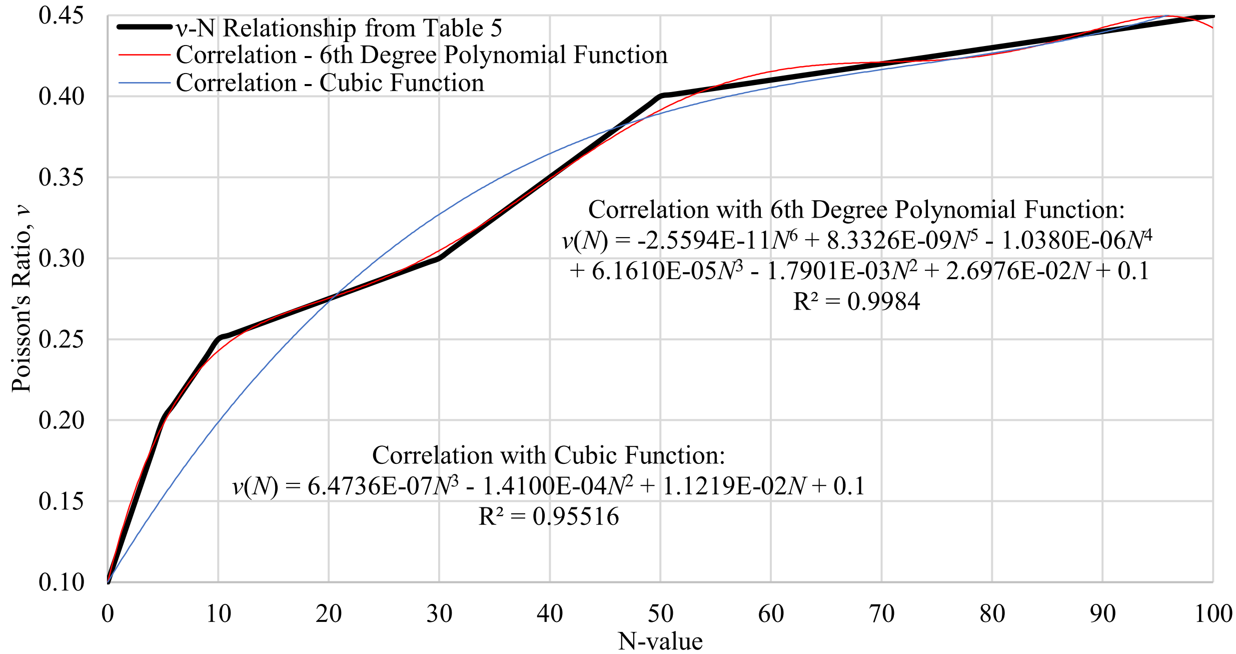

Figure 3.4. Correlations between ν and N-value based on the Data of Table 3.2.

For determination of *E* of soils in drained condition, correlations between the *E* and N-value of cohesionless soils of Table 2.6 (Ohya et al., 1982; Briaud et al., 1985; Mayne & Frost, 1989; Kulhawy & Mayne, 1990; Yagiz et al., 2008; Bozbey & Togrol, 2010; Kenmogne et al., 2011; Cheshomi & Ghodrati, 2015; Anwar, 2018) were considered. Since the cavity expansion of borehole of PMT with an inflatable pressure meter cell is considered to have a similar mechanism to the one of HDD bore with drilling mud pressure, PMT-based correlations were preferred for the collection of Table 2.6. For a similar reason, DMT-based correlation of Mayne and Frost (1989) was also included to the collection of Table 2.6. By using the relationship between the *E* and E_D of Equation [2.13], the correlation of Mayne and Frost (1989) could be expressed in terms of *E* (instead of E_D) as presented in Equation [3.3].

$$E = 22Pa(1 - \nu^2)N^{0.82} \qquad [3.3]$$

With inclusion of the Equation [3.3], the correlations of Table 2.6 could be plotted as presented in Figure 3.5 for comparisons.

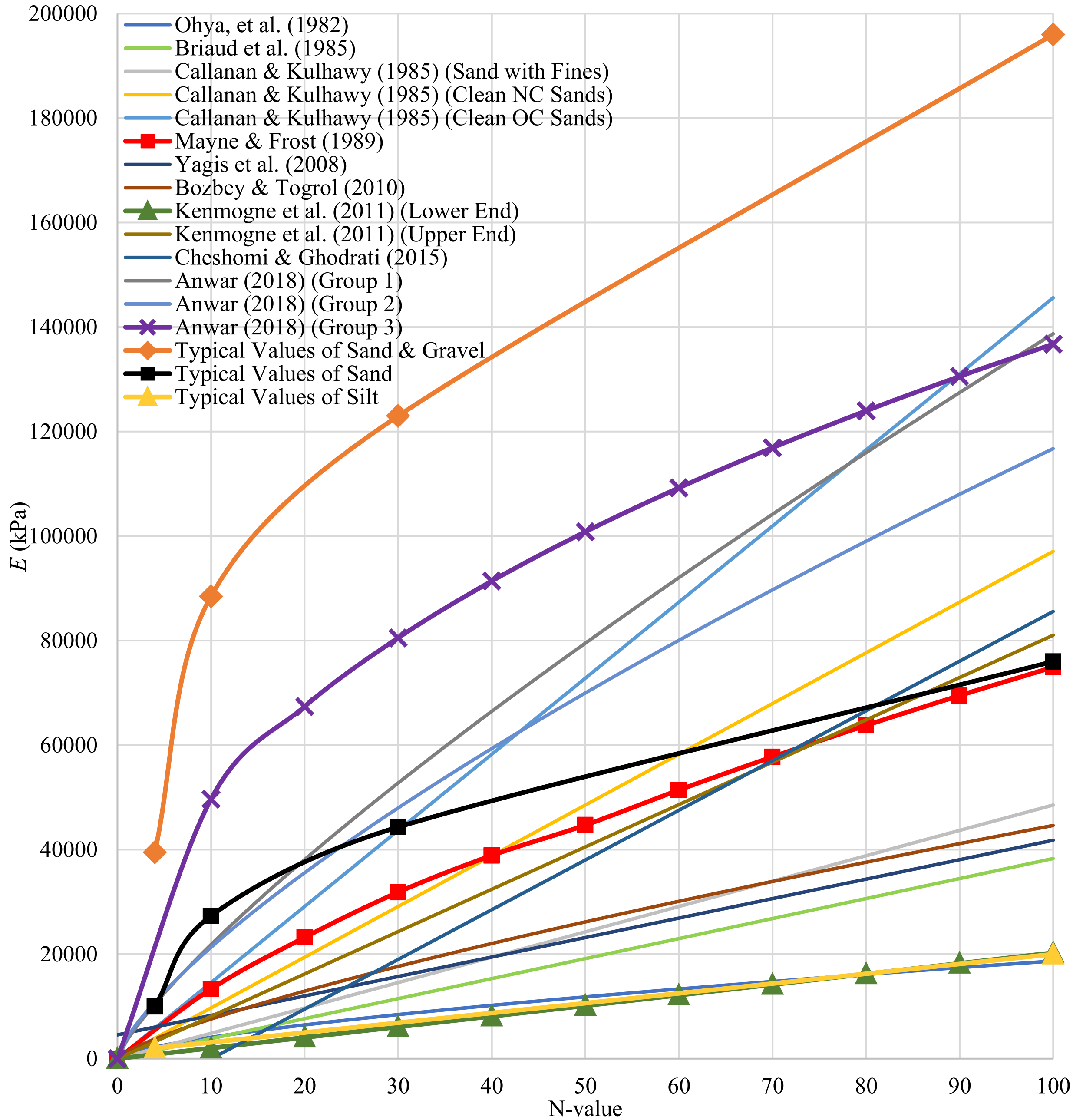

Figure 3.5. Correlations between the E and N-value of Various Cohesionless Soils from Table 2.6

A large scatter was observed from the relationship between the *E* and N-value of Figure 3.5. In order to select the most representable correlation for development of drained soil model of SPT-based method, validation of the correlations of Table 2.6 using the actual measurements of *E* was required. For the validation of correlations of Table 2.6, typical values of *E* presented in Table 2.7 (Bowles, 1996; Asperger and Bennett ,2011; and Das and Sobhan, 2013) were considered. The averages of the suggested values of *E* of Table 2.7 were plotted together with the correlations of Table 2.6 in Figure 3.5. From the comparison between the *E* estimations from correlations of Table 2.6 and suggestions of *E* values of Table 2.7, the most representable correlations for various types of cohesionless soils were found to be: soil group 3 of Anwar (2018) for sand and gravel, Mayne and Frost (1989) for sand, and "Lower End" of Kenmogne et al. (2011) for silt, silty sand, or fine sand. For sand and gravel, it is important to understand the composition of this soil type can be varied greatly, which may cause significant reduction of accuracy of estimated *E*. For fine sands, there are some correlations found by Briaud et al. (1985), Callanan and Kulhawy (1985), Yagiz et al. (2008), and Bozbey and Togrol (2010); however, compared to the values of *E* suggested by Asperger and Bennett (2011), all of these correlations tend to make overprediction; therefore, the "Lower End" correlation of Kenmogne et al. (2011), which estimates relatively conservative *E*, may be more applicable instead. Using the relationship between the *G* and *E* of Equation [2.11], *G* of cohesionless soils can be defined as presented in Table 3.3.

Table 3.3. Selected Relationships between the Shear Modulus and N-value of Cohesionless Soils

Soil Description	Author	Year	*G* (kPa)
Sand and Gravel	Anwar	2018	$G = \frac{89.07 P_a N^{0.4398}}{(1+\nu)}$
Sand	Mayne & Frost	1989	$G = 11Pa(1-\nu)N^{0.82}$
Silt, Silty Sand, or Fine Sand	Kenmogne et al.	2011	$G = \frac{P_a N}{(1+\nu)}$

Another option considered for determination of in-situ *G* is using the correlations between G_{max} found from seismic geophysical survey and N-value (Imai and Yoshimura, 1970; Ohba and Toriumi, 1970; Ohta et al., 1972; Ohsaki and Iwasaki, 1973; Imai and Tonouchi, 1982; Seed et al., 1983; Kramer, 1996; and Anbazhagan and Sitharam, 2010), which are presented in Table 2.9. For comparison between the elastic shear modulus approach and seismic geophysical survey approach, correlations for *G* in Table 2.6 and the ones for G_{max} in Table 2.9 were plotted together in Figure 3.6. Compared to the correlations for *G* in Table 2.6, the ones for G_{max} in Table 2.9 are found much greater.

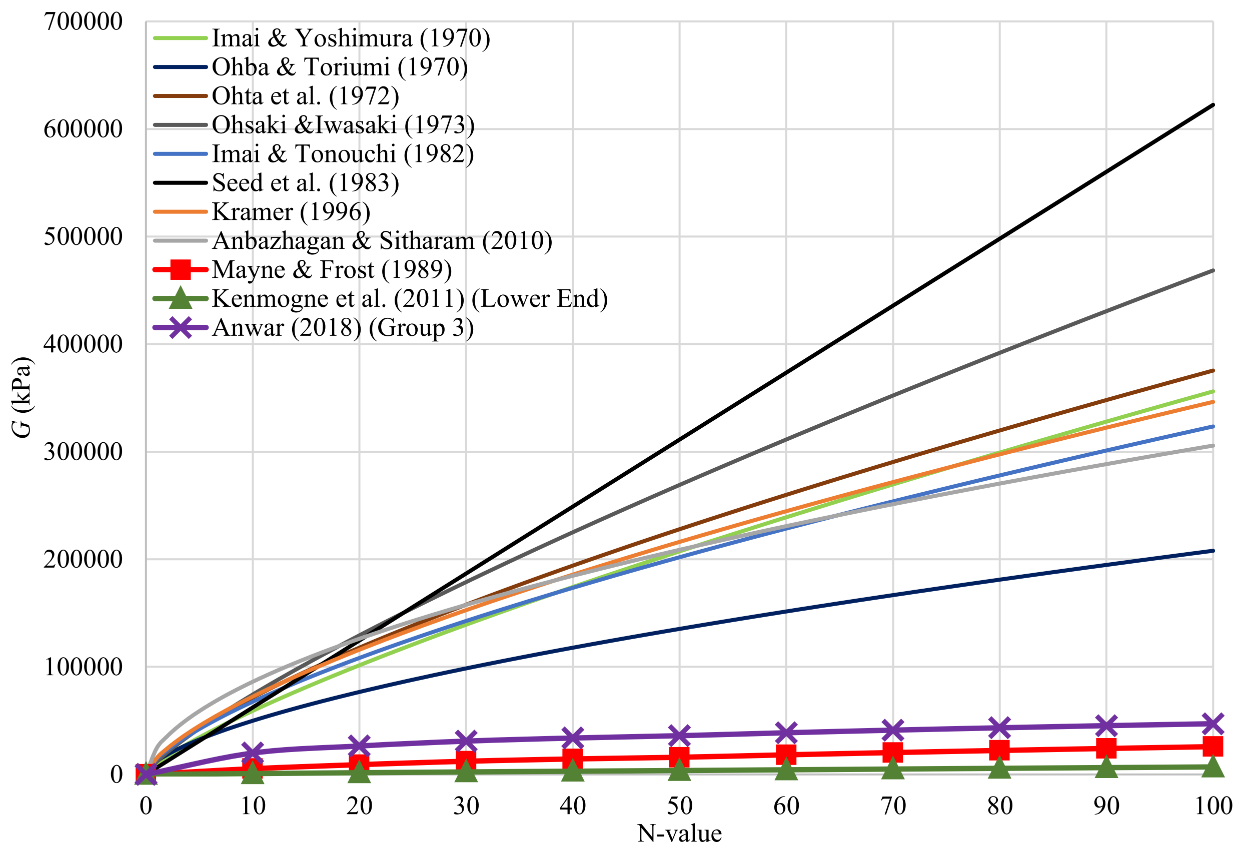

Figure 3.6. Comparison between the Correlations of G of Table 2.6 and Correlations of G_{max} of Table 2.9

As it is mentioned in Chapter 2, G is significantly reduced as γ increases (Hardin and Drnevich, 1972); therefore, determination of representable G for P_{max} prediction of SPT-based method requires γ of HDD bore at hydrofracture to be known. However, current studies related to hydrofracture of HDD are limited to measurements of drilling mud pressure using downhole pressure transducers, which do not include the measurements of deformation of bores. According to Table 2.10, typical range of strain of triaxial compression tests with cohesionless soils (sand and gravel, silt and sand) under low confining pressure was found from 2×10^{-3} to 5×10^{-3} % (Weissman and Hart, 1961). By substituting 3.5×10^{-3} % (average between 2×10^{-3} % and 5×10^{-3} %) into γ and 3×10^{-4} % (suggestion of Kramer (1996) for reference shear strain) into γ_r of Equation [2.14], G_{max} of correlations of Table 2.9 can be reduced as presented in Figure 3.7. From Figure 3.7, it can be seen the reduced G_{max} values have reasonable agreement with the elastic G values obtained using correlations of Table 2.6. Since the both elastic modulus approach and seismic geophysical survey approach have good agreement, correlations of Table 2.6 were considered to be valid for determination of G of drained soil model of SPT-based method.

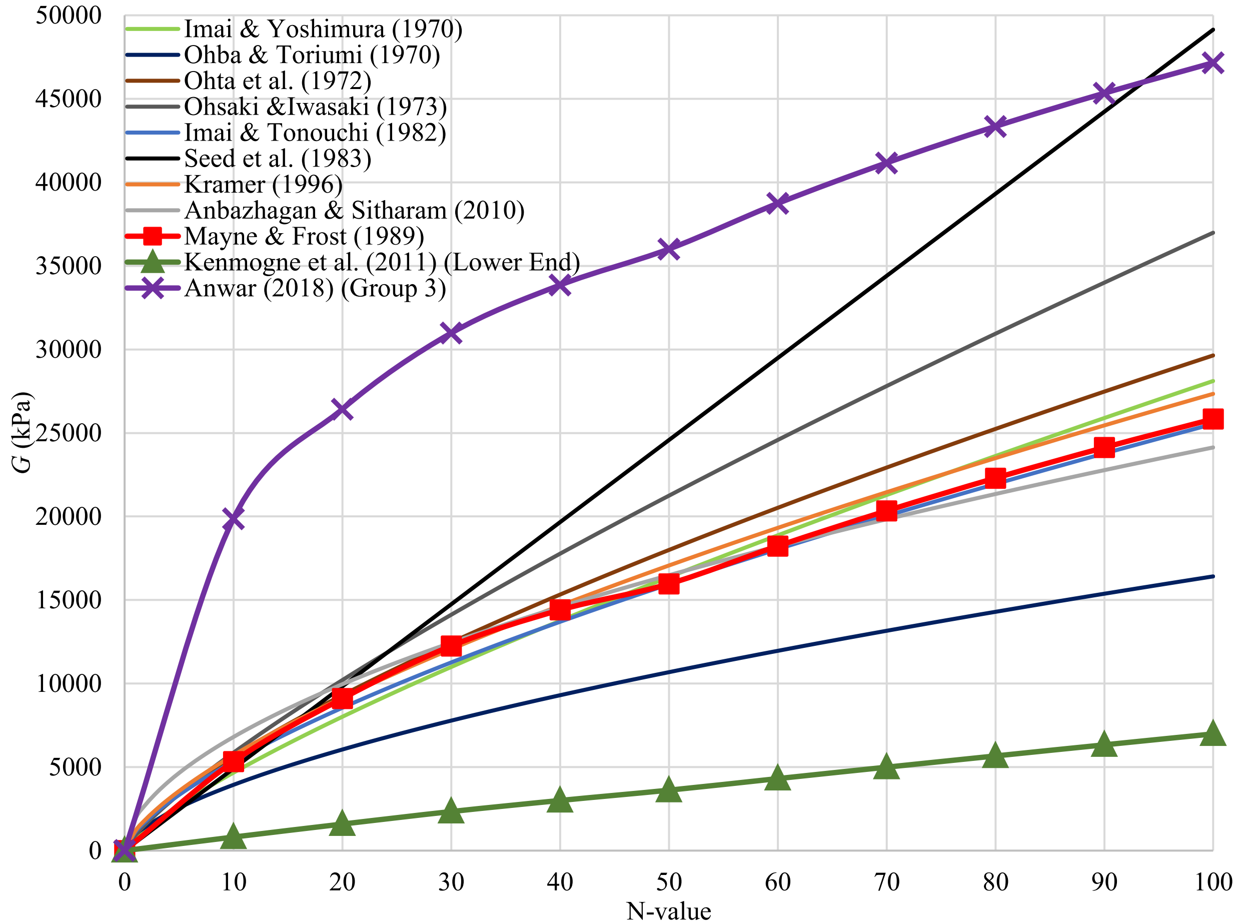

Figure 3.7. Comparison between the G correlations of Table 2.6 and Reduced G_{max} correlations of Table 2.9

3.2.3. Undrained Soil Model

Fine-grained soils with large clay contents typically have extremely small pore spaces, which makes the soils nearly impermeable. When an external load is applied to such impermeable soil at saturated condition, the volume of soil is reduced by the volume of dissipated pore water, and this phenomenon is termed as consolidation. Due to the low permeabilities of fine-grained soils, consolidation generally happens over a much longer timeframe compared to the duration of HDD; therefore, fine-grained soils are often assumed to remain undrained during HDD operations. In the undrained state, stress induced from pressurized drilling mud directly translates to the pressure in pore water, which pushes soil grains further away from each other. As long the pressure remains in pore water without being dissipated, soil cannot mobilize friction between its grains; hence, it is valid to assume its internal friction angle, φ, being nearly zero. While mobilization of internal friction is absent, the shear strength of undrained fine-grained soil is governed by the bonds formed with interaction between the pore water and soil particles, especially clay minerals. The strength of these bonds of fine-grained soil is termed as cohesion, c, or undrained shear strength, S_u. By substituting zero for φ and replacing c with the undrained shear strength, S_u, Equation [2.1] can be simplified as presented in Equation [3.4]. This equation is often referred as the New Orleans Method.

$$P_{max} = \sigma'_0 + u + S_u \tag{3.4}$$

3.2.3.1. Limitation of SPT with Fine-Grained Soils

As mentioned previously, SPT results taken at depths of less than 2 m are unreliable (de Mello, 1971; and Schmertmann, 1975) and they should not be used for any type of design. Since the consisting grains are bound together by cohesive bonding, fine-grained soils with large clay content are expected to have less grain movement from the energy transferred from blow of SPT hammer, compared to the coarse-grained soils with low fine contents those are generally known to be cohesionless. However, confinement at such shallow depths is not likely to be sufficient to restrict the deformation of surcharging strata, resulting the higher degrees of freedom. Therefore, the energy transferred from the blow of the SPT hammer may be partially lost to the deformation of surcharging strata, and only the remainder energy will contribute to shearing of soil at the tip of split spoon sampler. Considering the difficulty with approximation of energy loss toward the deformation of surcharging strata, SPT results from such shallow depths are not reliable for fine-grained soils as well. Moreover, suggestion for the value of C_R of Equation [2.2] at depths shallower than 3 m is not available; hence, SPT result from depths less than 3 m would also not be applicable for the standardization of N-value (N_{60}).

SPT is also known to be inapplicable for measurement of strength of fully softened clayey soil with high water content. Strength of this type of soil is often found extremely low, which is not even capable of withstanding the deadweight of SPT hammer. As a common practice in geotechnical investigation, any N-value for fine-grained soil of less than ten is considered to be unreliable, since the strength of the soil is too low for accurate measurement using SPT. For such cases of clayey soils with very low strength, measurement of S_u using an in-situ VST is recommended. Since VST can be conducted with the same drilling rig as SPT and it can be conveniently switched from SPT by simply swapping the split spoon sampler to the vane shear apparatus, it is highly recommended to take both SPT and VST measurements when soft clayey soil is encountered. According to the writer's experience, most drilling contractors had shear vane apparatus available and switching from SPT to VST could be done without having significant time loss. Once the measurement of S_u is obtained from in-situ VST, it is important to make a correction using soil plasticity data as determined from Atterberg limit testing in the laboratory. Detailed information related to the correction of S_u can be found in ASTM D2573. If the strength of in-situ clayey soil exceeds the capability of the conventional two inches using a four inch vane, smaller vanes with greater torque measurement devices are recommended to be used. However, depending on the drilling contractors, those smaller vanes might not be always available. CPT also provides reliable measurement of S_u and it is often used as another option. Nevertheless, since the CPT requires a different type of rig from SPT and the main focus of Chapter 3 is on the development of an SPT-based method, detailed information about CPT will not be further discussed.

3.2.3.2. Undrained Shear Strength, S_u

As mentioned previously, the behavior of fine-grained soil is governed by the interaction between the pore water and soil particles, which has a complicated mechanism involving multiple factors, such as in-situ water content, plasticity, sensitivity, stress history, and etc. For capturing all of these characteristics and taking precise strength measurement of such complicated material, information collected from SPT may not be sufficient. Especially for the S_u of fully softened clayey soil with high water content, measurement from SPT is unreliable. Despite the doubtful applicability of N-value for the estimation of S_u of fine-grained soil, it is still thought there is somewhat proportional relationship between the S_u and N-value, as the SPT conducted on a soft soil results a small N-value; meanwhile, the one conducted on a relatively stiffer soil is expected to result a larger N-value. Therefore, many researchers (Sanglerat, 1972; Terzaghi and Peck, 1967; Hara et al. 1974; and others) believed that N-value is still valid to be considered as an indicator of stiffness of fine-grained soil, and they continued finding the relationship between the S_u and N-value over the years. The 36 correlations listed in Table 2.11 are the outcomes of that research, and these correlations are also plotted in Figure 3.8 for comparison.

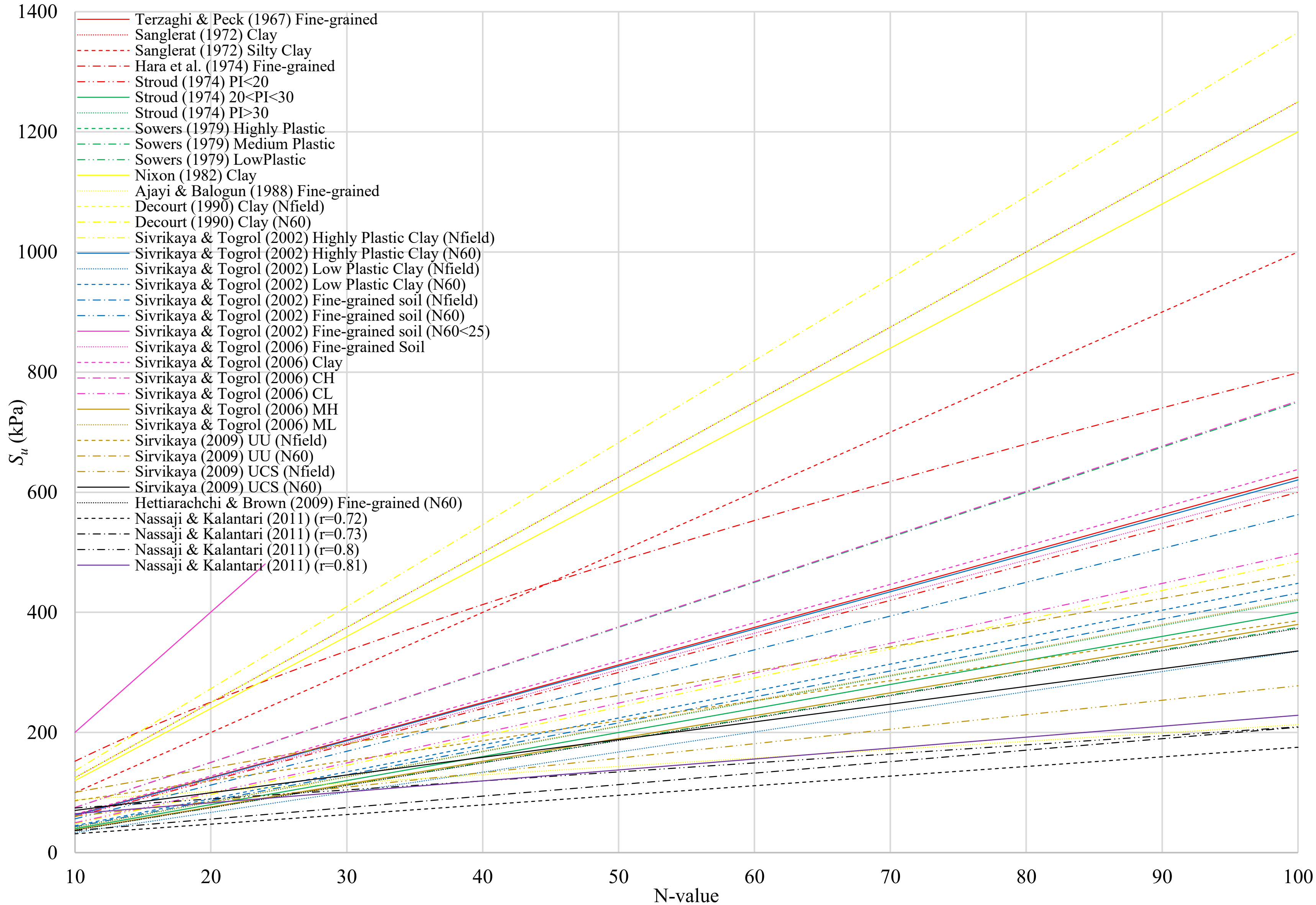

Figure 3.8. Correlations between S_u and N-value of Various Fine-Grained Soils from Table 2.11

Since the recently developed correlations (Sivrikaya and Togrol, 2002; Sivrikaya, 2009; and Nassaji and Kalantari, 2011) include more variables compared to the classical correlations, it is not possible to present all 36 correlations of Table 2.11 in the same two-dimensional plot. For reduction of number of variables of the recently developed correlations to simplify them as the functions with a single independent variable, N-value, a logical assumption for the plasticity parameters: *w%*, *LL*, and *PI*, was required. From the sensitivity analyses of plasticity parameters, *PI* was found to be the most sensitive variable; moreover, proportional relationships between the S_u and *PI* were found from the recently developed correlations. Considering the findings of sensitivity analyses, clayey soil with low plasticity, Kaolinite, was expected to result more conservative values of S_u with the recently developed correlations. For the presentation of all 36 correlations of Table 2.11 depicted in Figure 3.8, typical values of Kaolinite: *w%* as 40%, *LL* as 65%, *PL* as 23%, and *PI* as 42%, were substituted into the recently developed correlations.

Relationship between the S_u and N-value depicted in Figure 3.8 shows a great scatter, and the scatter tends to become even greater as N-value increases (Compared to the difference between the highest and lowest estimations of S_u for N-value of 10 is 163 kPa, the one for N-value of 100 is 1191 kPa). For development of undrained soil model of SPT-based method, it was required to determine a representative relationship between the S_u and N-value that captures the overall trend of all 36 correlations. In order to do so, maximum, minimum, and average values of S_u were considered, as plotted in Figure 3.9; however, both of the maximum and minimum values of S_u for N-value of 100 were 1366 kPa and 175 kPa for fine-grained soils, which were thought to be too extreme and unrealistic.

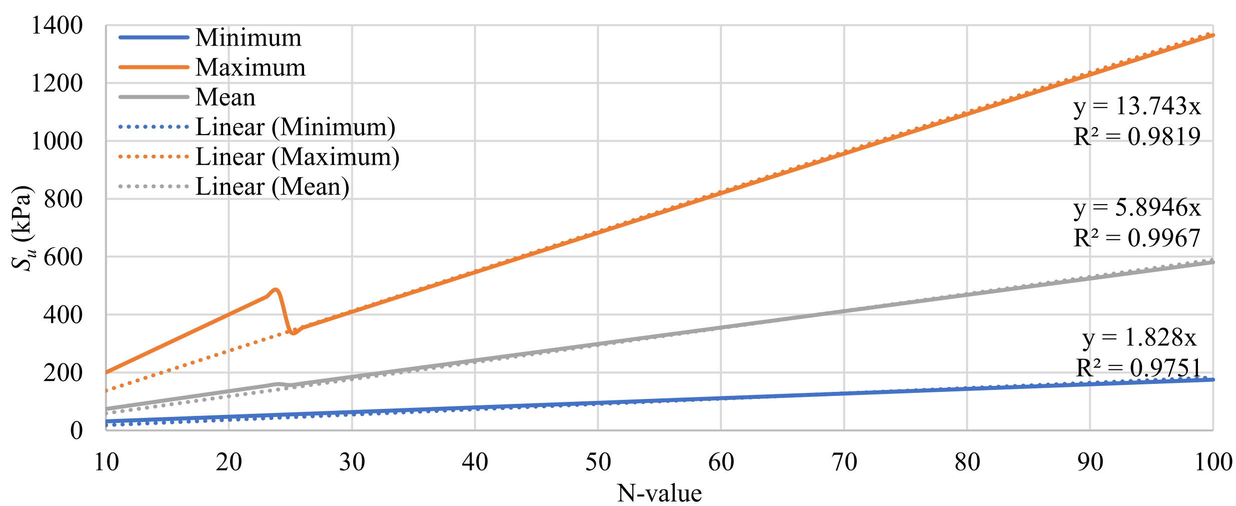

Figure 3.9. Maximum, Minimum, and Average Values of Undrained Shear Strength, S_u

From Figure 3.9, average values of S_u could be defined in terms of N-value and atmospheric pressure, P_a, as presented in Equation [3.5].

$$\frac{S_u}{P_a} = 0.058946N \qquad [3.5]$$

where: P_a is 100 kPa.

The estimation of S_u using Equation [3.5] for an N-value of 100 is 581 kPa, and this estimation seems to be a reasonably conservative. Fine-grained soil with such high N-value is typically heavily overconsolidated glacial till, and for this type of very hard soil, 581 kPa as S_u is not considered to be excessively high. Coincidently, Equation [3.5] was found almost identical to the correlation between the S_u and N_{60} reported by Terzaghi et al. (1996), which is presented in Equation [3.6].

$$\frac{S_u}{P_a} = 0.06N_{60} \qquad [3.6]$$

Some geotechnical engineers have suggested that the correlation of Terzaghi et al. (1996) can be used as a lower bound model during informal discussions. Considering the great scatter in the relationship between S_u and N-value, the correlation reported by Terzaghi et al. (1996) seems to be a reasonably conservative choice for development of undrained soil model.

For the estimation of S_u of hard clayey soils, it is also important know if the soils contain any discontinuities. Typical example of hard clayey soils, heavily overconsolidated glacial till, often contain small or large-sized cracks inside its mass, due to its brittle behaviors. These discontinuities are problematic, because they may cause significant reduction of S_u of soil mass; therefore, even though the intact piece of soil is found strong, the entire soil mass containing discontinuities may be much weaker. Most of these problematic discontinuities are naturally weathered joints, and they are visually distinguishable from mechanical joints, which are created during the augering process of site investigation. Typically, mechanical joints do not have eroded surfaces and they have relatively sharper edges, compared to the naturally weathered joints. Hence, if hard clayey soil is encountered during the site investigation, it was highly recommended to check if there is any crack found inside the soil mass, and leave detailed descriptions about the cracks in borehole log, so the estimation of S_u can be adjusted during design accordingly.

3.2.4. Factor of Safety

As mentioned previously, numerous researchers (Keulen, 2001; USACE, 2007; Elwood, 2008; Xia, 2009; Staheli et al., 2010; Rostami, 2017; Goerz et al., 2019; Miller and Robinson, 2019; and others) have commented on a tendency of the Delft method to overpredict P_{max}. Even with the suggestion of van Brussel and Hergarden (1997) on reduction of $R_{p,max}$ ($H/2$ for clayey/peat layers, and $2/3\ H$ in sand layer), overprediction of P_{max} of Delft method was still reported as presented in Table 3.4. Considering the tendency of overprediction, majority of researchers and designers suggest a minimum factor of safety of 2.0. In awareness of risks involving damages from hurricane and storms, USACE required a stricter minimum factor of safety of 3.0 for undrained soil conditions for design guideline of New Orleans District.

Table 3.4. Overprediction of P_{max} of Delft Method Reported by Multiple Researchers (Keulen, 2001; Elwood, 2008; Xia, 2009; Rostami, 2017; and Goerz et al., 2019)

Author	Year	Source of Pressure Measurement Data		Soil Type	Overprediction Reported by Researchers
Keulen	2001	Field and laboratory measurements		Sands	53 % (Calculated based on the provided data)
Elwood	2008	Laboratory experiment	Small-scale test	Compacted hydrosands	N/A (Not Compared)
			Large-scale test		15 % (Test data of layered soil was excluded)
Xia	2009	Data of Elwood (2008)	Small-scale tests		30 % to 80 %
			Large-scale test		150 %
		Laboratory experiment	Large-scale test		160 % to 190 %
Rostami	2017	Combined data of Elwood (2008) and Xia (2009)			105 %
Goerz et al.	2019	Downhole pressure measurements of hydrofracture occurrences over 30 HDD projects		Sands and Clays	63 %

3.2.5. Design Algorithm

By organizing both drained and undrained soil models developed in previous sections, SPT-based method could be presented in a simplified form of flow chart as shown in Figure 3.10.

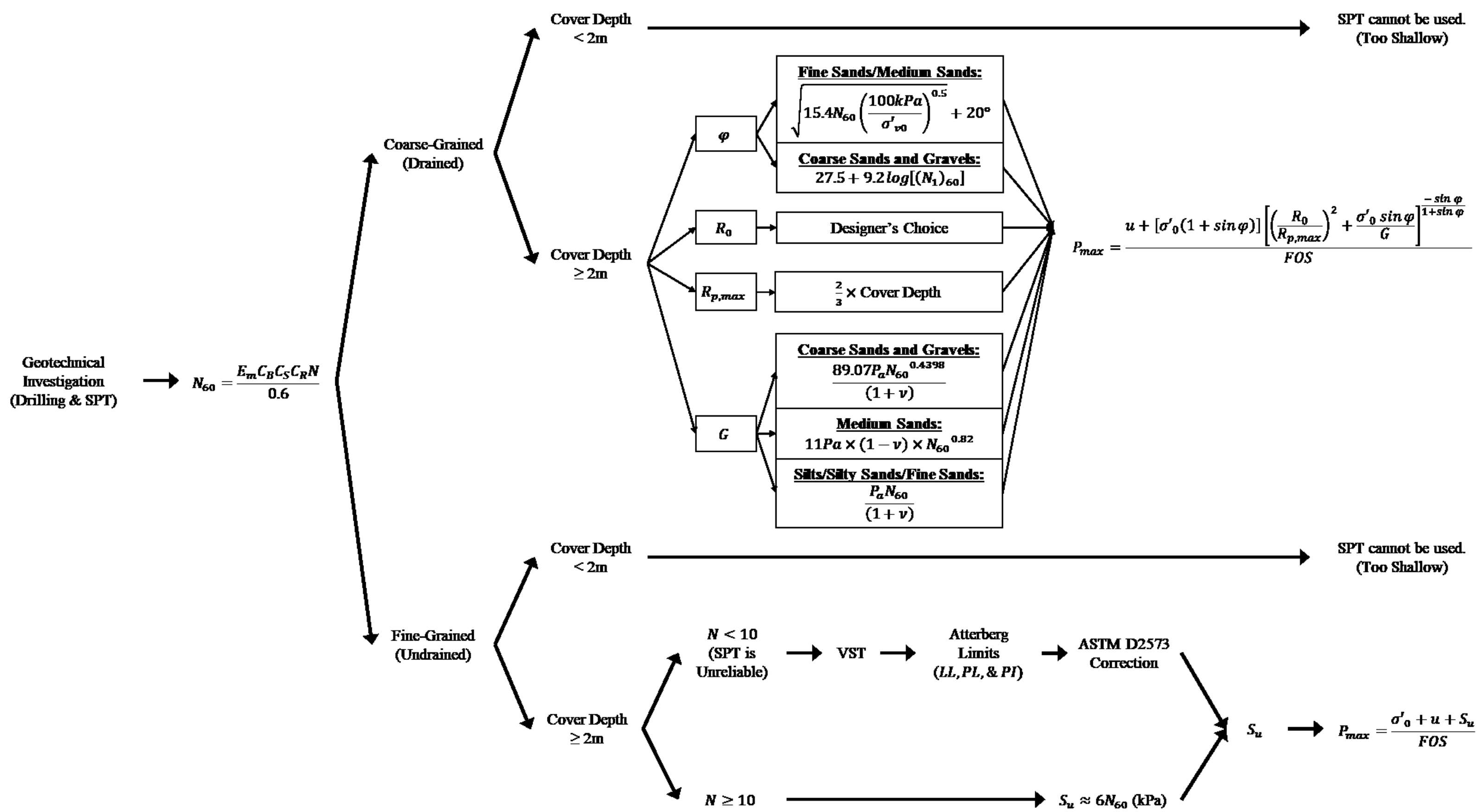

Figure 3.10. Simplified Algorithm of the SPT-based Method.

3.3. RESULTS

With all the input parameters defined, results of the SPT-based method for both drained and undrained soils are plotted in Figure 3.11, 3.12, 3.13 and 3.14, respectively to present the relationship between the P_{max} and N-value. To focus on the effect of variation in N-value, P_{max} was normalized by σ'_0 for both the drained and undrained soil models. Since the risk of hydrofracture is known to be the greatest for pilot hole drilling, diameter of borehole, D_0, is set from 4 inches to 8 inches. For the rest of the parameters, settings of Table 3.5 were applied.

Table 3.5. Settings Applied for the Drained and Undrained Soil Models

	Drained Soil Model	Undrained Soil Model
Saturated Soil Density	2200 kg/m^3	2000 kg/m^3
Depth of Groundwater	0m (Up to the ground surface)	
Type of SPT Hammer	USA safety hammer (2 turns on cathead)	

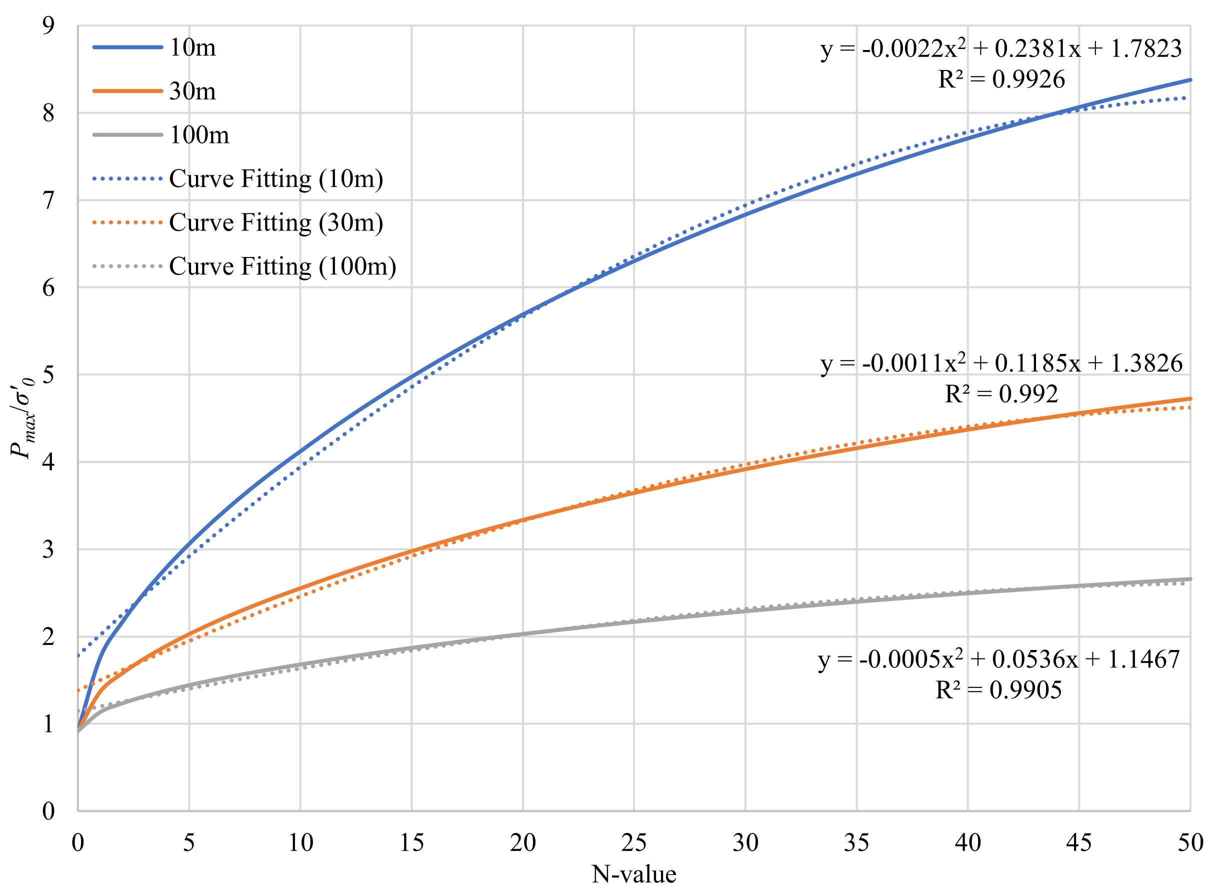

Figure 3.11. Relationship between the P_{max}/σ'_0 and N-value for 4" Diameter Borehole for Drained Soil Model

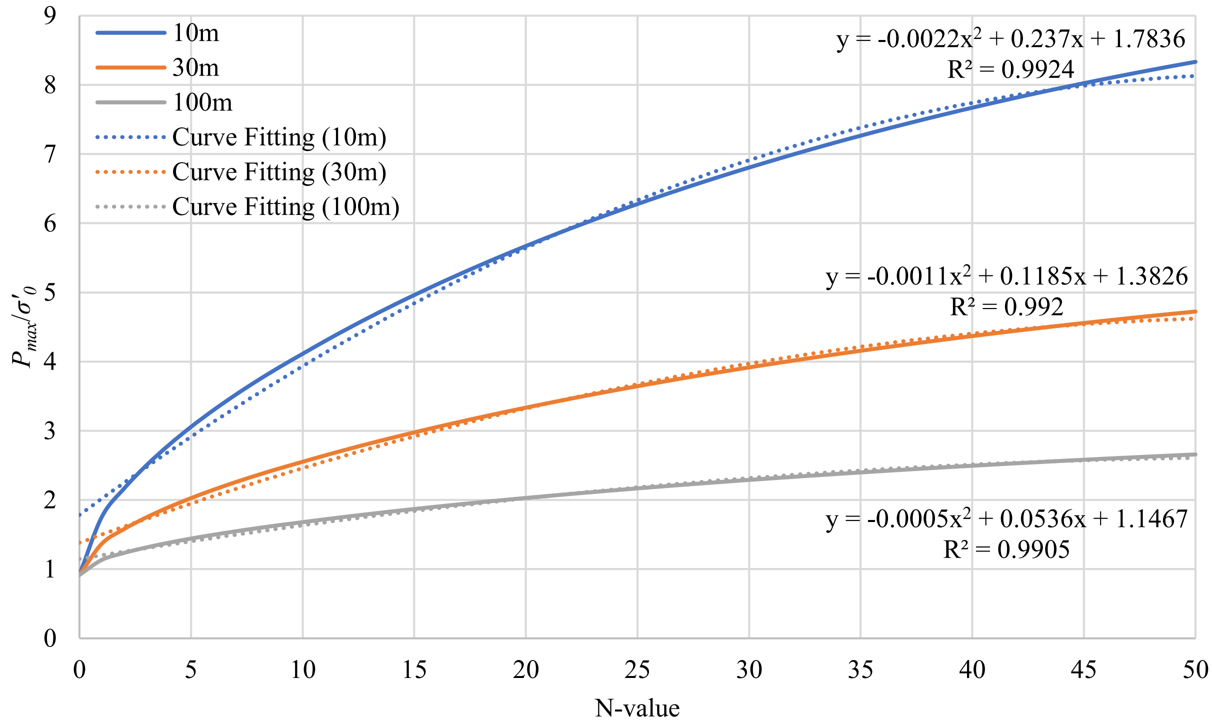

Figure 3.12. Relationship between P_{max}/σ'_0 and N-value for 6" Diameter Borehole for Drained Soil Model

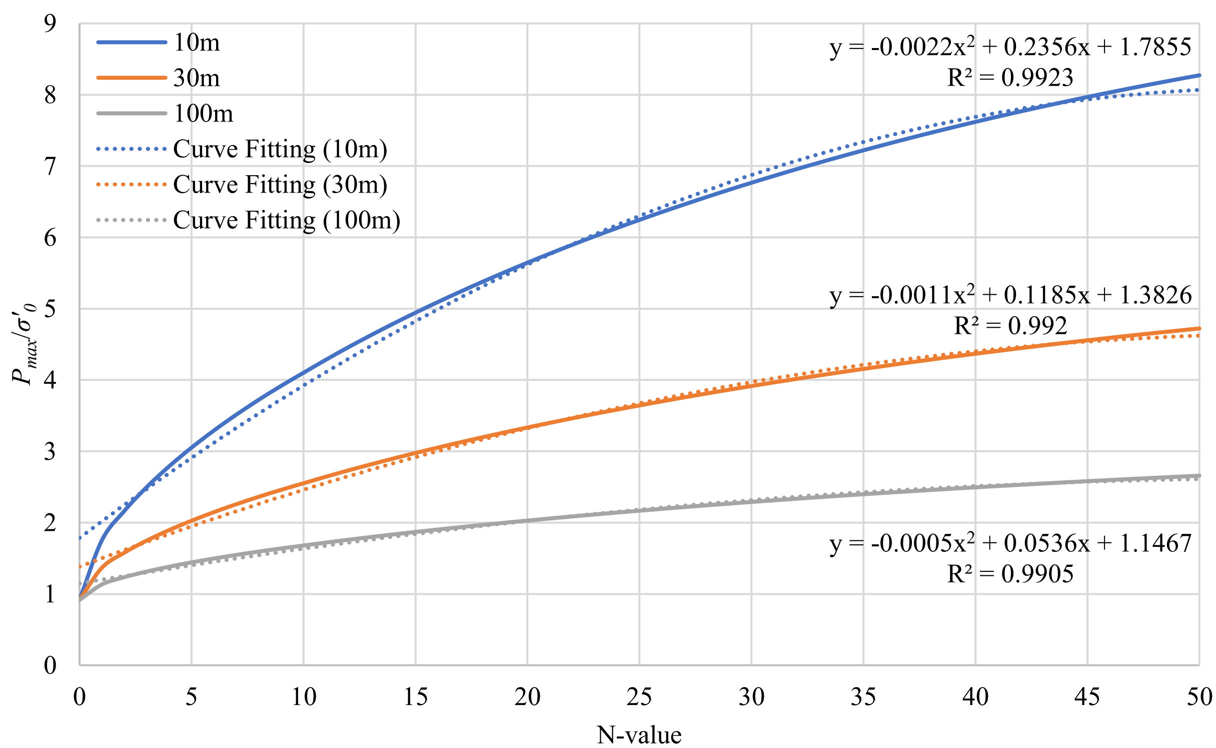

Figure 3.13. Relationship between P_{max}/σ'_0 and N-value for 8" Diameter Borehole for Drained Soil Model

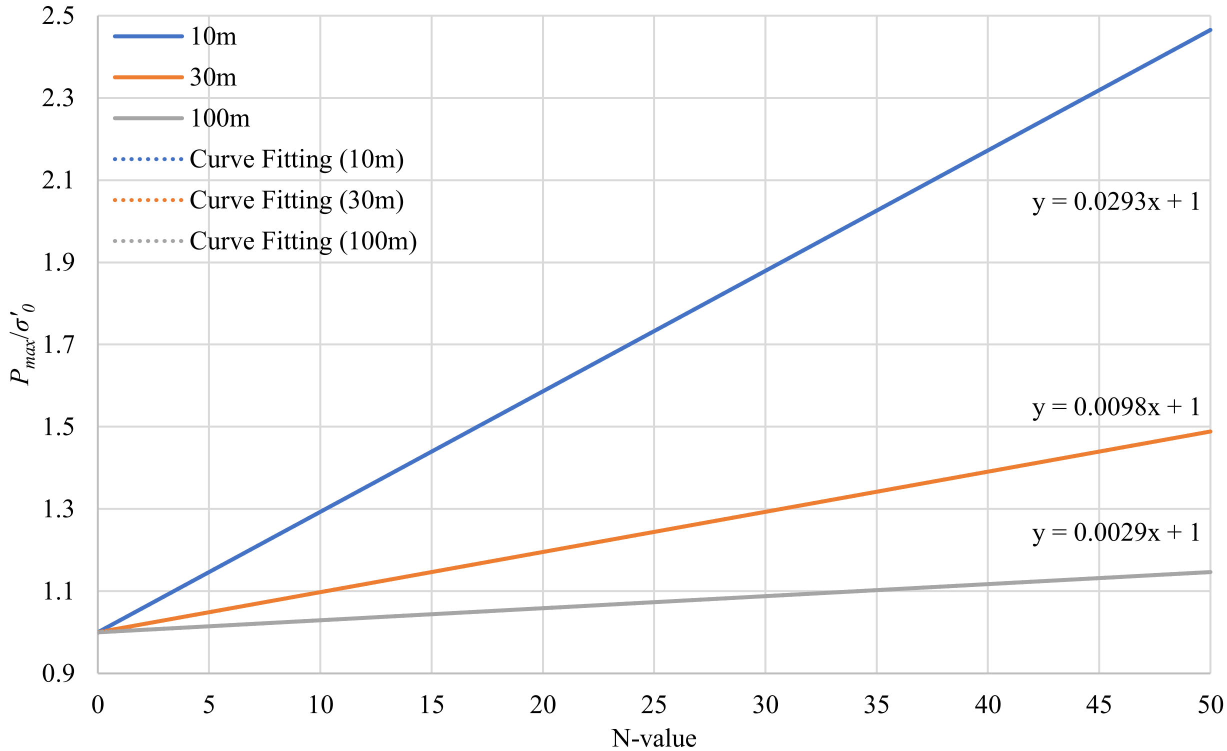

Figure 3.14. Relationship between P_{max}/σ'_0 and N-value for Undrained Soil Model

From comparisons between Figures 3.11, 3.12, and 3.13 of drained soil model, variation in D_0 was found to have a negligible effect on the estimated P_{max}. Since the depths of HDD bores are typically much greater than the diameters of pilot holes for most cases, $(R_0/R_{p,max})^2$ of Equation [2.1] becomes negligible (van Brussel and Hergarden (1997) suggested definition of $R_{p,max}$ in terms of cover depth: $R_{p,max} = H/2$ for clayey/peat layers, and $R_{p,max} = 2/3\ H$ in sand layer); subsequently, the resulting P_{max} becomes independent of D_0. In particular, the undrained soil model is entirely independent from D_0, as the P_{max} of Equation [3.4] is equal to the summation of total stress and undrained shear strength (for undrained soils, $\phi = 0°$ was applied to Equation [2.1]; hence, the term including $(R_0/R_{p,max})^2$ was cancelled), and the undrained soil model could simply be presented in a single plot, as shown in Figure 3.14. Compared to the undrained soil model, which exhibits a linear relationship between P_{max}/σ'_0 and the N-value, the drained soil model shows a non-linear relationship. This non-linear relationship of drained soil model had good agreement with quadratic function ($R^2 > 0.99$). The estimated P_{max} of drained soil model was found to be much greater than the ones for undrained soil model; moreover, the N-value was found to be more sensitive on the estimation of P_{max} at shallower depths for both drained and undrained soil models.

3.4. VALIDATIONS

To validate the prediction of P_{max} based on the newly developed SPT-based method, field and laboratory measurements of hydrofracture pressure from multiple literature sources (Luger and Hergarden, 1988; Keulen, 2001; Xia, 2009; Staheli et al., 2010; and Lan and Moore, 2018) were reviewed.

3.4.1. Case Studies of Staheli et al. (2010)

In this section, a case study provided by Staheli et al. (2010) with field measurements of downhole pressure was presented for the demonstration of application of SPT-based method. For the case study, downhole pressure was monitored and recorded with pressure transducer during the HDD operation, and any significant and immediate spikes from pressure reading were remarked, which are the potential occurrences of hydrofracture. This case study provides hydrofracture data from two different locations, and the site geometry and soil parameters of each location are presented in Table 3.6.

Table 3.6. Site Geometry and Soil Parameters (Staheli et al., 2010)

Input Parameters		**Location 1**	**Location 2**
Soil description		Hard to very hard silt with sand	Medium dense sands with some silt and gravel
Depth of the bore below ground surface	h_s	21 m	10 m
Height of groundwater over the bore	h_w	7.6 m	10 m
Bore radius	R_0	0.1524 m	0.1524 m
Unit weight of soil above the groundwater	γ	19.42 kN/m^3	20.42 kN/m^3
Soil friction angle	φ	30°	28°
Cohesion	c	24 kPa	0
Poisson's ratio	v	0.35	0.30
Modulus of elasticity	E	23940 kPa	11970 kPa
Shear modulus	G	8858 kPa	4597 kPa

Staheli et al. (2010) explained that the soil parameters of Table 3.6 were either provided from geotechnical reports or estimated based on published values; however, they did not include any further details about the process for determination of soil parameters. Without any borehole log or SPT results (especially the N-value) available, the SPT-based method was not directly applicable. Although the N-value was not provided, a pseudo-N-value could be approximated with back-calculation on the basis of the information given in Table 3.6. Once the pseudo-N-value was back-calculated, it was substituted into drained and undrained soil models of SPT-based method for the estimation of P_{max}. Lastly, the estimated P_{max} was compared with the measurement of hydrofracture pressure to check the validity of the SPT-based method.

3.4.1.1. Location 1

As presented in Table 3.6, soil found from location 1 is “hard to very hard silt with sand”. Since the major component of this soil is silt and the description of consistency is “hard to very hard”, which is typically used for strong clayey soils, this material may contain significant amount of fine grains; therefore, borehole surrounded with this type of soil is assumed to be undrained during HDD. Terzaghi and Peck (1967) suggested describing the consistency of cohesive soil with N_{60} greater than 30 as “hard”. Assuming 50 as N_{60} for “very hard” soil, N_{60} of 40, which is an average between \30 and 50, is considered to be reasonable. With the substitution of 40 for N_{60} of Equation [3.6], S_u of this soil can be determined to be 240 kPa. The initial effective stress, σ'_0, of Location 1 can be calculated as 338 kPa, as presented in Equation [3.7].

$$\sigma'_0 = \gamma \times (h_s - h_w) + (\gamma - \gamma_w) \times h_w \qquad [3.7]$$

The initial in-situ pore pressure, u, can be calculated as 75 kPa as presented in Equation [3.8].

$$u = \gamma_w h_w \qquad [3.8]$$

By substituting 338 kPa, 75 kPa, and 2.0 for σ'_0, u, and FOS in Equation [3.4] of undrained soil model, the P_{max} of Location 1 can be estimated to be 327 kPa. According to Staheli et al. (2010), the measurement of hydrofracture pressure at Location 1 was 56 psi, which is equivalent to 386 kPa; therefore, the estimation of P_{max} of undrained soil model is reasonably close to the measurement of hydrofracture pressure with a conservative margin of safety.

3.4.1.2. Location 2

As presented in Table 3.6, soil found from Location 1 is “medium dense sands with some silt and gravel”. Since the major component of this soil is sand, silt, and gravel, this material is considered to be coarse-grained soil; therefore, the borehole is assumed to be drained during HDD. Terzaghi and Peck (1967) suggested describing the relative density of cohesionless soil with N_{60} between 10 and 30 as "medium dense". To approximate the pseudo-N-value of this soil, φ of 28° and E of 11970 kPa from Table 3.6 were used in the back-calculation. Considering the surcharging effect on φ of soil, back-calculation of pseudo-N-value requires σ'_0 to be known, which can be calculated as 106.1 kPa using Equation [3.7]. With substitution of 28° and 106.1 kPa into φ and σ'_0 of Equation [3.2], pseudo-N-value can be back-calculated as 4.28 as presented in Equation [3.9].

$$N_{60} = \frac{(\varphi - 20°)^2}{15.4} \times \sqrt{\frac{\sigma'_0}{100\text{kPa}}} \qquad [3.9]$$

Prior to the approximation of pseudo-N-value using the correlation between the E and N-value of Equation [3.3], v is required to be known. By introducing the cubic correlation of Figure 3.4 into the definition of v of Equation [3.3], pseudo-N-value can be back-calculated as 8.23 as presented in Equation [3.10].

$$E = 22Pa\left[1 - \left(6.4736 \times 10^{-7} {N_{60}}^3 - 1.4100 \times 10^{-4} {N_{60}}^2 + 1.1219 \times 10^{-2} N_{60} + 0.1\right)^2\right] {N_{60}}^{0.82} \qquad [3.10]$$

Considering the suggested interval of N_{60} for "medium dense" cohesionless soil, which is from 10 to 30, back-calculated N-value of 4.28 is excessively small. Since the back-calculated N-value of 8.23 is reasonably close to the lower boundary N_{60} of "medium dense soil", 10, the pseudo-N-value of the given soil is approximated as 8. With substitution of 8 and 106.1 kPa into N_{60} and σ'_0 of Equation [3.2], φ can be determined as 30°. With substitution of 8 into the sand correlation of Mayne and Frost (1989) of Table 3.3, G can be determined as 4378 kPa. By substituting 106.1 kPa, 98 kPa, 30°, 4378 kPa, and 2.0 into σ'_0, u, φ, G, and FOS of Equation [3.1] of drained soil model, P_{max} of Location 2 can be estimated as 395 kPa. According to Staheli et al. (2010), the measurement of hydrofracture pressure at Location 2 was 55 psi, which is equivalent to 379 kPa. The estimation of P_{max} of drained soil model is slightly higher than the measurement of hydrofracture pressure; however, they are still reasonably close to each other.

3.4.2. Validations with Case Studies from Multiple Literatures

In a similar manner, SPT-based method was applied for the case studies from multiple literature reports (Luger and Hergarden, 1988; Keulen, 2001; Xia, 2009; Staheli et al., 2010; and Lan and Moore, 2018) as presented in Table 3.7. Estimations of P_{max} using SPT-based method without FOS and measurements of hydrofracture pressure were plotted together in Figure 3.15; from the correlation between them, it was found the SPT-based method overpredicts by 85%.

Table 3.7. Estimations of P_{max} of SPT-based Method and Measurements of Hydrofracture Pressure using the Case Studies from Multiple Literatures (Luger and Hergarden, 1988; Keulen, 2001; Xia, 2009; Staheli et al., 2010; and Lan and Moore, 2018)

<table>
<tr><th>Author</th><th>Year</th><th colspan="2">Type of Measurements</th><th>Estimated (kPa)</th><th>Measured (kPa)</th></tr>
<tr><td>Luger & Hergarden</td><td>1988</td><td colspan="2">Field Measurement</td><td>98</td><td>320</td></tr>
<tr><td rowspan="6">Keulen</td><td rowspan="6">2001</td><td colspan="2">Field Measurement (BTL 21)</td><td>172</td><td>450</td></tr>
<tr><td colspan="2">Laboratory Measurement (BTL 48)</td><td>544</td><td>370 to 400</td></tr>
<tr><td rowspan="4">Field Measurement (BTL 47)</td><td>1st</td><td>274</td><td>210</td></tr>
<tr><td>2nd</td><td>274</td><td>200</td></tr>
<tr><td>3rd</td><td>126</td><td>50</td></tr>
<tr><td>5th</td><td>44</td><td>20</td></tr>
<tr><td rowspan="15">Xia
(Based on the Data of Elwood, 2008)</td><td rowspan="15">2009</td><td rowspan="10">Laboratory Measurement (Small-scale test)</td><td>T1</td><td>130</td><td>135</td></tr>
<tr><td>T2</td><td>166</td><td>143</td></tr>
<tr><td>T3</td><td>195</td><td>165</td></tr>
<tr><td>T4</td><td>195</td><td>186</td></tr>
<tr><td>T5</td><td>207</td><td>190</td></tr>
<tr><td>T6</td><td>219</td><td>184</td></tr>
<tr><td>T7</td><td>219</td><td>219</td></tr>
<tr><td>T8</td><td>239</td><td>261</td></tr>
<tr><td>T9</td><td>258</td><td>215</td></tr>
<tr><td>T10</td><td>304</td><td>286</td></tr>
<tr><td rowspan="5">Laboratory Measurement (Large-scale test)</td><td>LS1</td><td>108</td><td>95</td></tr>
<tr><td>LS2</td><td>108</td><td>81</td></tr>
<tr><td>LS3</td><td>108</td><td>78</td></tr>
<tr><td>LS4</td><td>108</td><td>78</td></tr>
<tr><td>LS5</td><td>111</td><td>151</td></tr>
<tr><td rowspan="2">Staheli et al.</td><td rowspan="2">2010</td><td rowspan="2">Field Measurement</td><td>Location 1</td><td>321</td><td>386</td></tr>
<tr><td>Location 2</td><td>395</td><td>379</td></tr>
<tr><td rowspan="5">Lan & Moore</td><td rowspan="5">2018</td><td rowspan="5">Laboratory Measurement (Large-scale test)</td><td>HB11</td><td>20</td><td>16</td></tr>
<tr><td>HB10</td><td>31</td><td>28</td></tr>
<tr><td>HB12</td><td>45</td><td>57</td></tr>
<tr><td>HB13</td><td>81</td><td>102</td></tr>
<tr><td>HB15</td><td>135</td><td>160</td></tr>
</table>

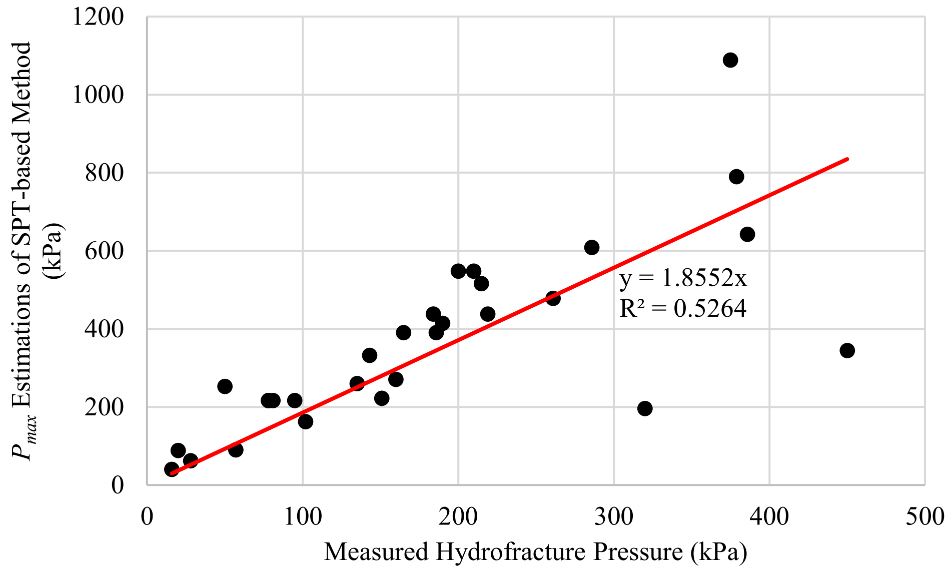

Figure 3.15. Relationship between the P_{max} estimated with SPT-based Method (Without the Application of FOS) and Measured Hydrofracture Pressure (Adopted Methodology from Rostami, 2017; and Goerz et al., 2019)

3.5. DISCUSSION

As presented in Table 3.4, a tendency of the Delft method to overpredict P_{max} was reported by multiple researchers (Keulen, 2001; Elwood, 2008; Xia, 2009; Rostami, 2017; and Goerz et al., 2019); moreover, the level of overprediction was found to be inconsistent. Since one of the major purposes of development of an SPT-based method was to improve the estimation of P_{max} with a systematic procedure of determination of geotechnical parameters, such inconsistency in overprediction was expected to be reduced using SPT-based method. To show the improvement in accuracy of estimation of P_{max} achieved through this study, the validation data in Table 3.7 was compared with the ones provided by other researchers (Keulen, 2001; Elwood, 2008; Xia, 2009; Rostami, 2017; and Goerz et al., 2019). For the comparison, multiple sets of data including P_{max} estimations of Delft method and measurements of hydrofracture pressure, $P_{Hydrofracture}$, were correlated as presented in Table 3.8. Linear functions, which were intended to pass through the origin, were adopted for development of the correlations. Coefficients of the linear functions, which are the ratios between the P_{max} and $P_{Hydrofracture}$, were presented with correlation strengths in terms of R^2. For visual comparisons, the correlations of Table 3.8 were also plotted as presented in Figure 3.16. Due to the outliers, strengths of some correlations were significantly low; therefore, correlations with data excluding the outliers were also included as references. Except the correlation found by Goerz et al. (2019) had high strength of R^2 = 0.9021, most of the ones found by the other researchers (Keulen, 2001; Elwood, 2008; Xia, 2009; and Rostami, 2017) had significantly low strengths (R^2 = -0.0620 from Keulen, 2001; R^2 = -165.3 from Elwood, 2008; R^2 = -0.2640 from Xia, 2009; and R^2 = 0.3248 from Rostami, 2017). Correlation found with the SPT-based method had a relatively higher strength of R^2 = 0.5264 than the other ones, and the strength could be increased up to 0.8644 by excluding two extreme outliers; therefore, the systematic procedure of determination of geotechnical parameters would have contributed to the improvement of accuracy of P_{max} estimation using the Delft method. Since most ratios between P_{max} and $P_{Hydrofracture}$ were greater than 2.0, application of a minimum factor of safety of 2.0 is strongly recommended for the SPT-based method. For the estimation of P_{max} in high-risk environments, a stricter minimum factor of safety of 3.0 suggested by USACE for New Orleans District may be required.

Table 3.8. Ratios between the P_{max} Estimation of Delft Method and Measured Hydrofracture Pressure, $P_{Hydrofracture}$, found by Multiple Researchers (Keulen, 2001; Elwood, 2008; Xia, 2009; Rostami, 2017; and Goerz et al., 2019)

Author	**Year**	**$P_{max}/P_{Hydrofracture}$ (With Outliers)**	**R^2 (With Outliers)**	**Number of Outliers**	**$P_{max}/P_{Hydrofracture}$ (Without Outliers)**	**R^2 (Without Outliers)**
Keulen	2001	1.5289	-0.0620	1	2.7306	0.9588
Elwood	2008	0.8679	-165.3	0	0.8679	-165.3
Xia	2009	1.5879	-0.2640	0	1.5879	-0.2640
Rostami	2017	2.0183	0.3248	1	1.7906	0.3875
Goerz et al.	2019	1.6259	0.9021	0	1.6259	0.9021
SPT-based Method		1.8522	0.5264	2	2.1946	0.8644

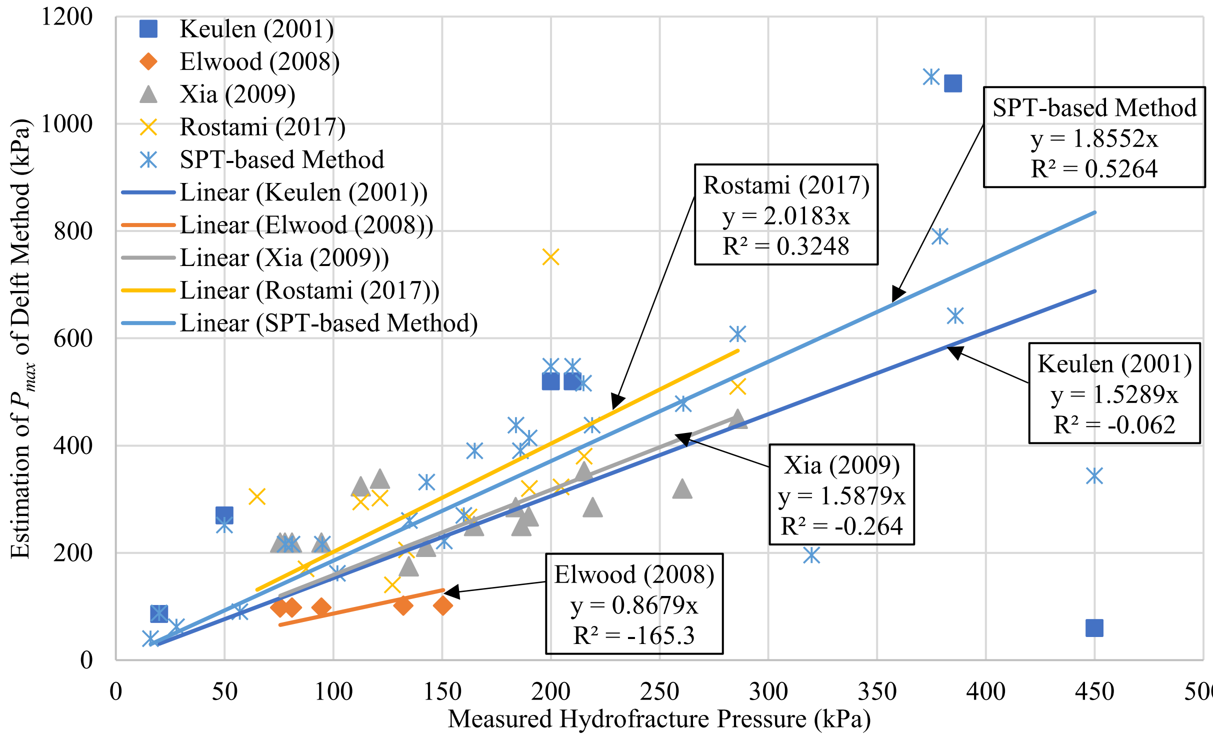

Figure 3.16. Correlations between the Estimations of P_{max} of Delft Method and Measured Hydrofracture Pressure based on the Data of Table 3.8

It is important to understand that the newly developed SPT-based method is a simplified approach for the estimation of P_{max} with a limited information of in-situ soil, and it is the most applicable during the preliminary design stage. Since the SPT-based method suggests geotechnical parameters to be determined with empirical correlations, it is definitely more economical than the other methods involving actual field/laboratory tests. However, it is almost impossible to expect the empirically determined geotechnical parameters to simulate all the complicated mechanism of soil, and they are not expected to be as reliable as the ones obtained from the actual field/laboratory tests. Therefore, in order to obtain the most accurate estimation of P_{max}, design engineers should try their best to conduct the actual field/laboratory tests as much as possible within the project allowances, instead of blindly relying on the SPT-based method. Borehole logs from site investigation reports include records of important characteristics of in-situ soils, which may be helpful for the analysis of complicated soil behaviors; hence, the information from borehole logs should be reflected to the N-value substituted into SPT-based method. For any region with unique characteristics of soils or geologies, it is strongly recommended to collect the relevant information from local experiences. Lastly, diligent monitoring of downhole mud pressure, and proper risk response mitigation strategies are the key components to lower the likelihood of hydrofracture occurrence and the impact of risk on the HDD project (Murray et al., 2013).

3.6. CONCLUSION

Through careful reviews and comparisons between correlation models for each individual geotechnical parameter within the Delft equation, an SPT-based method for the estimation of maximum allowable mud pressure was developed for both drained and undrained bore conditions. The SPT-based method provides systematic procedures for determination of geotechnical parameters; moreover, it allows direct application of N-value for the estimation of P_{max}.

While the undrained soil model presented a simple linear relationship between the normalized P_{max} (with respect to initial effective stress) and N-value, drained soil model presented non-linear relationship. Compared to P_{max} estimated using the undrained soil model, the one estimated with the drained soil model was found to be much greater for the same N-value.

Based on the data including measurements of hydrofracture pressure from multiple literature reports (Keulen, 2001; Elwood, 2008; Xia, 2009; and Rostami, 2017), the estimations of P_{max} made with the SPT-based method were found to be more accurate compared to the ones made by other researchers. Considering the results from validation, a minimum factor of safety of 2.0 is strongly recommended for the SPT-based method.

In future, the SPT-based method will be further validated using additional hydrofracture pressure measurements for improvement of the accuracy of the model, and the existing concept of safety factor may be redefined in terms of risk level associated with each HDD project. Moreover, it would be worthwhile to make a comparison between the undrained soil model of SPT-based method and the Queen’s method (Xia, 2009) developed for soft clayey soils. Since the Queen’s method has become popular among the practitioners over past decade and it addresses an important concept about the possibility of tensile failure of clays, the Queen’s method may be considered as a candidate for the undrained soil model of SPT-based method, instead of conserving the Delft method.

REFERENCES

Abu-Hejleh, N., O'Neill, M.W., Hanneman, D., and Atwoll, W.J., (2003), "*Improvement of the Geotechnical Axial Design Methodology for Colorado's Drilled Shafts Socketed in Weak Rock,*" Report No. CDOT-DTD-R-2003-6, Colorado Department of Transportation, Denver, 192 p.

Ajayi, L.A. and Balogun, L.A. (1988), "Penetration Testing in Tropical Lateritic and Residual Soils – Nigerian Experience," *Proceedings of the First International Symposium on Penetration Testing*, ISOPT-1, Orlando, March 20-24, 1998, Vol. 1, pp. 315-328.

Anbazhagan, P., and Sitharam, T.G., (2010), "Relationship between Low Strain Shear Modulus and Standard Penetration Test 'N' values." *ASTM Geotechnical Testing Journal*, Vol. 33, No. 2, pp. 150-164.

Anwar, M.B., (2018), "Correlation between PMT and SPT Results for Calcareous Soil," *Housing and Building National Research Center Journal*, Vol. 14, No. 1, pp. 50-55.

API, (2009), – *Recommended Practice on the Rheology and Hydraulics of Oil-Well Drilling Fluids*, API RP 13D, Washington, D.C.

Asperger, M., and Bennett, D., (2011), "The Search for G: Selecting Shear Modulus for Use in Hydrofracture Analysis," *Proceedings of 2011 No - Dig Conference*, Washington, D.C., March 27-31, 2011

Baroid, (1998), – *Baroid Fluids Handbook*, Chapter 9: Rheology and Hydraulics, Baroid Drilling Fluids, Inc., Haliburton, Houston, Texas.

Barthélemy, H.C., (1974), "An Investigation of Relationships between the SPT and the Cone Penetrometer Resistance in Sands," Project Report, Department of Civil Engineering, Duke University.

Bozbey, I., and Togrol, E., (2010), “Correlation of Standard Penetration Test and Pressuremeter Data: A Case Study from Istanbul, Turkey.” *Bulletin of Engineering Geology and the Enbrivironment*, Vol. 69, pp. 505–515.

Bowles, J.E., (1996), – *Foundation Analysis and Design*, The McGraw Hill Companies, Inc., Fifth Edition, New York

Briaud, J.L., Noubani, A., Kilgore, A., and Tucker, L.M., (1985), “Correlation between Pressuremeter Data and Other Parameters,” Research Report, Civil Engineering, Texas A&M University, (Cited from Briaud, J.L., The Pressuremeter, Balkema, A.A., Rotterdam, 1992).

Brown, D.A., Turner, J.P., and Castelli, R.J., (2010), – *Drilled Shafts: Construction Procedures and LRFD Design Methods, NHI Course No. 132014 Geotechnical Engineering Circular NO. 10*, Federal Highway Administration, Report No. FHWA NHI-10-016, Washington, DC.

Callanan, J.F., and Kulhawy, F.H., (1985), ―Evaluation of Procedures for Predicting Foundation Uplift Movements‖, Report EL-4107, Electric Power Research Institute, Palo Alto, Aug. 1985, 124p.

Canadian Geotechnical Society, (2007), – *Canadian Foundation Engineering Manual*, Canadian Geotechnical Society, Fourth Edition, Richmond, British Columbia, Canada

Chen, J.-R., (2004), “Axial Behavior of Drilled Shafts in Gravelly Soils,” Ph.D. Dissertation, Cornell University, Ithaca, N.Y.

Cheshomi, A., and Ghodrati, M., (2015), “Estimating Menard Pressuremeter Modulus and Limit Pressure from SPT in Silty Sand and Silty Clay Soils. A Case Study in Mashhad, Iran,” *Geomechanics and Geoengineering*, Vol. 10, No. 3, pp. 194-202.

Clayton, C.R.I., (1990), “SPT Energy Transmission: Theory, Measurement and Significance,” *Ground Engineering*, Vol. 23, No. 10, pp. 35-43.

Das, B.M., and Sobhan, K., (2013), – *Principles of Geotechnical Engineering*, 8th Ed., Cengage Learning, Stamford, 2013, 362p.

Décourt, L., (1989), “The Standard Penetration Test State-of-the-Art Report,” *International Conference on Soil Mechanics and Foundation Engineering*, Rio de Janeiro.

De Mello, V.F.B., (1971), “The Standard Penetration Test,” *4th Panamerican Conference on Soil Mechanics and Foundation Engineering*, San Juan, Puerto Rico, June 1971, Vol. 1, pp. 1-86.

Elwood, D.E.Y., (2008), “Hydraulic Fracture Experiments in a Frictional Material and Approximation for Maximum Allowable Mud Pressure,” M.Sc. Thesis, Queens University, Kingston, Ontario, Canada, July 2008

Gibbs, H.J., and Holtz, W.G., (1956), “Research on Determining the Density of Sands by Spoon Penetration Testing,” *Proceedings of the 4th International Conference on Soil Mechanics*, London, UK, 1957, Vol. 1, pp. 35-39.

Goerz, S., Boelhouwer, N., and Taylor, J., (2019), “A New Perspective in Hydrofracture Analysis,” *Proceedings of 2019 No-Dig Conference*, Chicago, Illinois, March 17-20, 2019

Hara, A., Ohta, T., Niwa, M., Tanaka, S., and Banno, T., (1974), “Shear Modulus and Shear Strength of Cohesive Soils,” *Soils and Foundation*, Vol. 14, No. 3, Sept. 1974, pp. 1-12.

Hardin, B.O., and Drnevich, V.P., (1970), "Shear Modulus and Damping in Soils: I, Measurement and Parameter Effects, II. Design Equations and Curves," Technical Reports UKY 27-70-CE 2 and 3, College of Engineering, University of Kentucky, Lexington, Kentucky, July.

Hatanaka, M., and Uchida, A., (1996), “Empirical Correlation Between Penetration Resistance and Internal Friction Angle of Sandy Soils,” *Soils and Foundations*, Vol. 36, No. 4, pp. 1-9.

Hettiarachchi, H. and Brown, T., (2009), “Use of SPT Blow Counts to Estimate Shear Strength Properties of Soils: Energy Balance Approach,” *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 135, No. 6, pp. 830-834.

Iai, S., and Kurata, E., (1991), “Pore Water Pressures and Ground Motions Measured during the 1987 Chiba-Toho-Oki Earthquake,” Note of the Prot and Harbour Research Institute, No. 718, pp. 1-18.

Imai, T., and Tonouchi, K., (1982), “Correlation of N-value with S-wave Velocity and Shear Modulus.” *In Proceedings of the 2nd European Symposium on Penetration Testing*, pp. 57-72.

Imai, T., and Yoshimura, Y., (1970), “Elastic Wave Velocity and Soil Properties in Soft Soil.” *Tsuchito-Kiso*, Vol. 18, No. 1, pp. 17-22.

Kenmogne, E., Martin, J.R., and Geofor, S.A., (2011), “Correlation Studies between SPT and Pressuremeter Tests,” *Proceedings of the 15th African Regional Conference on Soil Mechanics and Geotechnical Engineering*, Mozambique, July 18-21, 2011

Kerisel, J., (1964), “Deep Foundations Basic Experimental Facts,” *Cimientos Profundos*, Mexico, Vol. 1, p. 5.

Keulen, B., (2001), “Maximum Allowable Pressures During Horizontal Directional Drillings Focused on Sand,” Master thesis, Delft University of Technology, Delft, Netherlands, September 2001

Kramer, S.L., (1996), – *Geotechnical earthquake engineering*, Prentice-Hall Inc., New Jersey

Kulhawy, F.H., (1969), “Finite Element Analysis of the Behavior of Embankments,” Ph.D. Dissertation, University of California, Berkeley, 1969, pp. 40-80.

Kulhawy, F.H. and Chen, J.-R. (2007), “Discussion of ‘Drilled Shaft Side Resistance in Gravelly Soils’ by Kyle M. Rollins, Robert J. Clayton, Rodney C. Mikesell, and Bradford C. Blaise,” *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 133, No. 10, pp. 1325-1328.

Kulhawy, F.H., Duncan, J.M., and Seed, H.B., (1969), “Finite Element Analysis of Stresses and Movements in Embankments during Construction,” Contract Report S-69-8, U.S. Army Engineers Waterways Experiment Station, Vicksburg, Nov. 1969, 169 p.

Kulhawy, F.H., and Mayne, P.W., (1990), – *Manual on Estimating Soil Properties for Foundation Design*, Electric Power Research Institute EL-6800, Project 1493-6, Electric Power Research Institute, Palo Alto, California.

Lake, L.M., (1974), “Discussion on Granular Materials,” *Proceedings of Conference on Settlement of Structures*, Cambridge, 1974, Pentech Press, pp. 663.

Lan, H., and Moore, I.D., (2018), “Practical Criteria for Borehole Instability in Sand during Horizontal Directional Drilling,” *Proceedings of 2018 No – Dig Conference*, Palm Springs, California, March 20-24, 2018

Liao, S.S.C., and Whitman, R.V., (1986), “Overburden Correction Factors for SPT in Sand,” *Journal of Geotechnical Engineering*, ASCE, Vol. 112, No. 3, pp. 373-337.

Marchetti, S., (1980), “In Situ Tests by Flat Dilatometer,” *Journal of Geotechnical Engineering Division*, ASCE, Vol. 106, No. GT3, Mar. 1980, pp. 299-321.

Mayne, P.W., and Frost, D.D., (1989), “Dilatometer Experience in Washington, D.C., and Vicinity,” *Transportation Research Record 1169*, Transportation Research Board, Washington, D.C., 1989, pp. 16-23.

Mayne, P.W., and Kemper, J.B., (1988), “Profiling OCR in Stiff Clays by CPT and SPT,” *ASTM Geotechnical Testing Journal*, Vol. 11, No. 2, pp. 139-147.

Meyerhof, G.G., (1956), “Penetration Tests and Bearing Capacity of Cohesionless Soils,” *Journal of Soil Mechanics and Foundation Division*, ASCE, Vol. 82, No. SM1, Jan. 1956, pp. 1-19.

Meyerhof, G.G., (1976), "Bearing Capacity and Settlement of Pile Foundations," *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 102, No. GT3, March, pp. 197-228.

Miller, M.A., and Robinson, J.L., (2019), “Soil Mechanics and Calculating Hydraulic Fracture Risk” *Proceedings of 2019 No-Dig Conference*, Chicago, Illinois, March 18-21, 2019

Murray, C.D., Osbak, M., and Bayat, A., (2013), “Horizontal Directional Drilling – Construction Risk Management Strategies,” *Proceedings of 2013 Pipelines Conference*, ASCE, Fort Worth, Texas, June 23-26, 2013

Nassaji, F., and Kalantari, B., (2011), “SPT capability to estimate undrained shear strength of fine-grained soils of Tehran, Iran,” *Electronic Journal of Geotechnical Engineering*, Vol. 16, 2011, pp. 1229-1238.

NEN, (2006), NEN 3650-1:2003+A1:2006. Eisen voor buisleidingsystemen - Deel 1: Algemene – Katern 1 tot en met 6 (Requirements for pipeline systems - Part 1: General – Quire 1 to 6), in English

NEN, (2017), NEN 3650-1+C1:2017. Eisen voor buisleidingsystemen - Deel 1: Algemene eisen (Requirements for pipeline systems - Part 1: General requirements), in English

Newcomb, D.E., and Birgisson, B., (1999), – NCHRP Synthesis of Highway Practice 278: Measuring In Situ Mechanical Properties of Pavement Subgrade Soils, Transportation Research Board, National Research Council, Washington D.C., 1999.

Nixon, I.K., (1982), "Standard penetration test: state of the art report," *Proceedings of the Second European Symposium on Penetration Testing*, Vol. 1, Amsterdam, 1982, pp. 3-24.

Ohba, S., and Toriumi, I., (1970), "Research on Vibration Characteristics of Soil Deposits in Osaka, Part 2, on Velocities of Wave Propagation and Predominant Periods of Soil Deposits." Abstract, Technical Meeting of Architectural Institute of Japan.

Ohsaki, Y., and Iwasaki, R., (1973), "On Dynamic Shear Moduli and Poisson's Ratio of Soil Deposits." *Soils and Foundations*, Vol. 13, No. 4, pp. 61-73.

Ohta, T., Hara, A., Niwa, M., and Sakano, T., (1972), "Elastic Moduli of Soil Deposits Estimated by N-values." *In Proceedings of the 7th Annual Conference, The Japanese Society of Soil Mechanics and Foundation Engineering*.

Ohya, S., Imai, T., and Matsubara, M., (1982), "Relationships between N-value by SPT and LLT measurement results," *Proceedings of the Second European Symposium on Penetration Testing*, Vol. 1, Amsterdam, 1982, pp. 125–130.

Olsen, R.S., (1997), "Cyclic Liquefaction based on the Cone Penetrometer Test," *Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils*, National Center for Earthquake Engineering Research, Dec. 1997, State University of New York at Buffalo, pp. 225-276.

O'Neill, M.W., and Reese, L.C., (1999), – *Drilled Shafts: Construction Procedures and Design Methods*. Publication No. FHWA-IF-99-025, Federal Highway Administration, Washington, D.C., p. 758.

Peck, R.B., Hanson, W.E., and Thorburn, T.H., (1974), – *Foundation Engineering*, John Wiley and Sons, Second Edition, New York

Poulos, H.G., (1978), "Normalized Deformation Parameters for Kaolin," Research Report R336, School of Civil Engineering, University of Sydney, pp. 18.

Riggs, C.O., (1986), “North American Standard Penetration Test Practice - An Essay,” *Proceedings of ASCE Conference on Use of In-situ Tests in Geotechnical Engineering*, Blacksburg, p. 945-965.

Rostami, A., (2017), “Prediction and Evaluation of Annular Pressure in Horizontal Directional Drilling,” Ph.D. Thesis, University of Alberta, Edmonton, Alberta, Canada, April 2017

Sabatini, P.J., Bachus, R.C., Mayne, P.W., Schneider, J.A., and Zettler, T.E., (2002), "Evaluation of Soil and Rock Properties," *Geotechnical Engineering Circular No. 5*, Federal Highway Administration, Washington, D.C., p. 385.

Sanglerat, G., (1972), – *The Penetration and Soil Exploration; Interpretation of Penetration Diagrams – Theory and Practice*, Elsevier Publishing Co, Amsterdam

Sanglerat, G., and Sanglerat, T.R.A., (1982), “Pitfalls of the SPT,” *Proceedings of 2nd Europe Symposium on Penetration Testing*, Amsterdam, May 24-27, 1982, Vol. 1, pp. 143-145.

Schmertmann, J.H., (1975), “Measurement of In-Situ Shear Strength,” *Proceedings of ASCE Specialty Conference on In-Situ Measurement of Soil Properties*, Vol. 2, Raleigh, 1975, pp. 57-138. (Closure: pp. 175-179)

Schmertmann, J. H., and Palacios, A., (1979), “Energy Dynamics of SPT,” *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 105, pp. 909-926.

Seed, H.B., and Idriss, I.M., (1970), “Soil Moduli and Damping Factors for Dynamic Response Analysis,” Report EERC70-10.

Seed, H.B., Idriss, I.M., and Arango, I., (1983), "Evaluation of Liquefaction Potential Using Field Performance Data," *Journal of the Geotechnical Engineering Division*, Vol. 109, No. 3, pp. 458-482.

Seed, H.B., Tokimatsu, K., Harder, L.F. and Chung, R.M., (1985), “Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations,” *ASCE Journal of Geotechnical Engineering*, Vol. 111, No. 12, pp. 1425-1445.

Sivrikaya, O., and Toğrol, E., (2002), “Relations between SPT-N and q_u,” *5th Intern. Congress on Advances in Civil Engineering*, Istanbul, Turkey, 2002, pp. 943–952.

Sivrikaya, O., and Toğrol, E., (2006), “Determination of undrained strength of fine-grained soils by means of SPT and its application in Turkey,” *Engineering Geology*, Vol. 86, No. 1, 2006, pp. 52-69.

Sivrikaya, O., (2009), “Comparison of Artificial Neural Networks Models with Correlative Works on Undrained Shear Strength,” *Eurasian Soil Science*, Vol. 42, No. 13, 2009, pp. 1487-1496.

Skempton, A.W., (1986), “Standard Penetration Test, Procedures and Effects in Sands of Overburden, Relative Density, Particle Size, Aging and Over-consolidation,” *Geotechnique*, Vol. 36, No.3, pp. 425-447.

Sowers, G.F., (1979), – *Introductory Soil Mechanics and Foundations: Geotechnical Engineering*, Macmillan, Fourth Edition, New York

Staheli, K., Bennett, D., O’Donnell, H.W., and Hurley, T.J., (1998), “Installation of Pipelines beneath Levees Using Horizontal Directional Drilling,” CPAR-GL-98-1, April 1998

Staheli, K., Price, C.G., and Wetter, L., (2010), “Effectiveness of Hydrofracture Prediction for HDD Design,” *Proceedings of 2010 No - Dig Conference*, Chicago, Illinois, May 2-7, 2010

Stroud, M.A., (1974), “The Standard Penetration Test in Insensitive Clays and Soft Rocks,” *Proceedings of the European Symposium on Penetration Testing*, Stockholm, June 5-7, 1974, Vol. 2, No. 2, pp. 367-375.

Tavenas, F.A., Capelle, J.F., and La Rochelle, P., (1970), "Étude des Sables submergés par échantillonnage non remanié," (Study of Submerged Sands by Undisturbed Sampling), *Canadian Geotechnical Journal*, Vol. 7, No. 1, p. 37.

Tavenas, F.A., (1971), "Discussion to de Mello (1971)," *4th Panamerican Conference on Soil Mechanics and Foundation Engineering*, Puerto Rico, Vol. III, pp. 64-69.

Terzaghi, K. and Peck, R.B., (1967), – *Soil Mechanics in Engineering Practice*, John Wiley and Sons, Second Edition, New York

Terzaghi, K., Peck, R.B., and Mesri, G., (1996), – *Soil Mechanics in Engineering Practice*, John Wiley and Sons, Third Edition, New York

Trautmann, C.H., and Kulhawy, F.H., (1987), "CUFAD – A Computer Program for Compression and Uplift Foundation Analysis and Design," Report EL-4540-CCM, Vol. 16, Electric Power Research Institute, Palo Alto, Oct. 1987, 148p.

Uchida, A., Kano, H., and Tokimatsu, K., (1990), "Correlation between Dynamic Properties and Static Properties of Sands obtained by In-Situ Freezing," *25th Annual Meeting of JSSMFE*, Vol. 1, pp. 767-768.

USACE, (2007), Hurricane and Storm Damage Reduction System Design Guidelines. Interim., U.S. Army Corps of Engineers, New Orleans District, October 23, 2007

Van Brussel, G.G., and Hergarden, H.J.A.M., (1997), "Research CPAR program: Installation of Pipelines beneath Levees Using Horizontal Directional Drilling," Department of Foundations and Underground Engineering, Delft Geotechnics, Delft, Netherlands, pp. 1-16.

Vesic, A.S., (1967), “A Study of Bearing Capacity of Deep Foundations,” Final Report, Georgia Institute of Technology, Atlanta, Georgia

Weissman, G.F., and Hart, R.R., (1961), "The Damping Capacity of Some Granular Soils," *Symposium on Soil Dynamics*, ASTM Special Technical Publication No. 305, June 1961, pp. 45-54.

Wroth, C.P., (1975), “In-Situ Measurement of Initial Stresses and Deformation Characteristics,” *Proceedings of ASCE Specialty Conference on In-Situ Measurement of Soil Properties*, Vol. 2, Raleigh, pp. 180-230.

Xia, H., (2009), “Investigation of Maximum Mud Pressure within Sand and Clay during Horizontal Directional Drilling,” Ph.D. Thesis, Queens University, Kingston, Ontario, Canada, January 2009

Yagiz, S., Akyol, E., and Sen, G., (2008), “Relationship between the Standard Penetration Test and the Pressuremeter Test on Sandy Silty Clays: A Case Study from Denizli.” *Bulletin of Engineering Geology and the Environment*, Vol. 67, pp. 405-410.

Yoshimi, Y., (1994), “Relationship among Liquefaction Resistance, SPT N-value and Relative Density for Undisturbed Samples of Sands,” Tsuchi-to-Kiso, JSSMFE, Ser. No. 435, Vol. 42, No. 4, pp. 63-37.

Yu, H.S., and Houlsby, G.T., (1991), “Finite Cavity Expansion in Dilatant Soils: Loading Analysis,” *Geotechnique*, 41, No. 2, pp. 173-183.

4. Factor of Safety for the Annular Pressure Design within Coarse-Grained Soils during Horizontal Directional Drilling

4.1. INTRODUCTION

As was explained in Chapter 2, various researchers (van Brussel and Hergarden, 1997; NEN, 2006; Elwood, 2008; Xia, 2009; Staheli et al., 2010; Rostami, 2017; Goerz et al., 2019, Miller and Robinson, 2019; and others) have attempted to improve the estimation of maximum allowable mud pressure, P_{max}, of Delft method (Luger and Hergarden, 1988), and some of them (van Brussel and Hergarden, 1997; Xia, 2009; Staheli et al., 2010; Goerz et al., 2019; Miller and Robinson, 2019; and others) were able to provide suggestions for the conservative application of Delft method. Currently, the suggestion of 2.0 as an additional factor of safety (FOS) for the P_{max} estimated with Delft method is the most commonly used approach among the practitioners of HDD industry. Another well-known suggestion is the one from Staheli et al. (2010), which allows estimating more conservative P_{max} by reducing the $R_{p,max}$ to be less than 2 to 3 times of bore diameter, and according to Miller and Robinson (2019), USACE Risk Management Center (RMC) supported this approach during the recent discussion. For P_{max} estimated with the reduced $R_{p,max}$ (Staheli et al., 2010), Miller and Robinson (2019) recommended applying 1.5 as an additional FOS. These additional FOS of 1.5 or 2.0 may allow more conservative estimations of P_{max}; however, these approaches with single value as FOS are not considered to be efficient or strategic, since they cannot flexibly handle all the hydrofracture risks associated with various HDD conditions. For example, HDD operations with different bore depths and soil strengths may have the levels of hydrofracture risks varied accordingly. While there is not any proper strategy available for the flexible handling of variation in risk levels, it is difficult to expect the hydrofracture risks of HDD to be efficiently mitigated. Therefore, instead of relying on a single value as FOS, it is preferable to establish a proper framework for the FOS, which may allow optimal estimation of P_{max} for the inadvertent return (IR) assessment of HDD.

In Chapter 4, a new concept about the framework for FOS of estimation of P_{max} is introduced. With consideration of causes of hydrofracture and the risk levels associated with various bore conditions, a framework to determine the FOS is proposed. P_{max} estimated with the proposed FOS framework was also validated using data from two case studies of Keulen (2001) and Staheli et al. (2010). Since the major scope of Chapter 4 is the introduction of newly proposed concept, the development of FOS framework is limited to coarse-grained soils for brevity.

4.2. METHODOLOGY

4.2.1. Base Model Selection – Back to Original Delft Method

Prior to construction of the FOS framework, it was required to establish a base model which is capable of making an estimation of P_{max}. By applying the FOS from the framework to the established based model, it was anticipated to secure sufficient margin of safety for the estimated P_{max}. For better understanding of practitioners, it was preferable to have a simple and well-known existing method as the base model of the FOS framework. As discussed previously, the Delft method developed by Luger and Hergarden (1988) has been the most commonly used method among the HDD industry over the past thirty years, and this method is capable of representing the cavity expansion phenomenon of HDD bore in a theoretical manner. Moreover, a significant number of researchers have worked for the improvement of Delft method. Therefore, by establishing the Delft method as a base model, the existing trend of IR assessment can be conserved, and an abrupt transition to lesser used methodologies can be avoided. The estimation of P_{max} using the Delft method is presented in Equation [4.1].

$$P_{max} = u + [\sigma'_0(1 + \sin\varphi) + c\cos\varphi + c\cot\varphi] \times \left[\left(\frac{R_0}{R_{p,max}}\right)^2 + \frac{\sigma'_0 \sin\varphi + c\cos\varphi}{G}\right]^{\frac{-\sin\varphi}{1+\sin\varphi}} - c\cot\varphi \qquad [4.1]$$

where: u is initial in-situ pore pressure, σ'_0 is initial effective stress, ϕ is internal friction angle, c is cohesion, R_0 is initial radius of the hole, $R_{p,max}$ is maximum allowable radius, and G is shear modulus

Currently, the most commonly used definition of $R_{p,max}$ is the suggestion of Delft Geotechnics (van Brussel and Hergarden, 1997), which are $H/2$ for clayey and peat soils, and $2/3\ H$ for sand. However, this suggestion of $R_{p,max}$ of Delft Geotechnics is known to have negligible effect on P_{max} (Staheli et al., 2010). In typical HDD projects, $R_{p,max}$ suggested by Delft Geotechnics: $H/2$ or $2/3\ H$, are much greater than R_0; therefore, it results $R_0/R_{p,max}$ as a small value. Subsequently, quadratic term of the Delft method, $(R_0/R_{p,max})^2$, becomes even much smaller, and this term approaches zero as H increases. Unless the HDD bore is located right underneath the ground surface, $(R_0/R_{p,max})^2$ remains a negligible term. While the term $(R_0/R_{p,max})^2$ remains negligible, reduction of $R_{p,max}$ from H to $H/2$ or $2/3\ H$ does not have significant effect on P_{max}. Therefore, despite the preference of suggestion of Delft Geotechnics on $R_{p,max}$ among the practitioners, it was thought to be better to keep the definition of $R_{p,max}$ simple, and to select the original Delft method with $R_{p,max}$ as H as the base model.

4.2.2. Factor of Safety Framework

The FOS framework is outlined in two major components, as presented in Figure 4.1. The first component of FOS framework is for correction factors, which are capable of converting the P_{max} of base model into the pressure equivalent to the actual hydrofracture pressure. The second component of FOS framework is for the risk-based design factors those relate to the consequences of hydrofracture during HDD. Therefore, a product between the correction factor of the first component and risk-based design factor of the second component is anticipated to be a FOS, which is capable of converting the P_{max} of the base model into P_{max} with sufficient margin of safety.

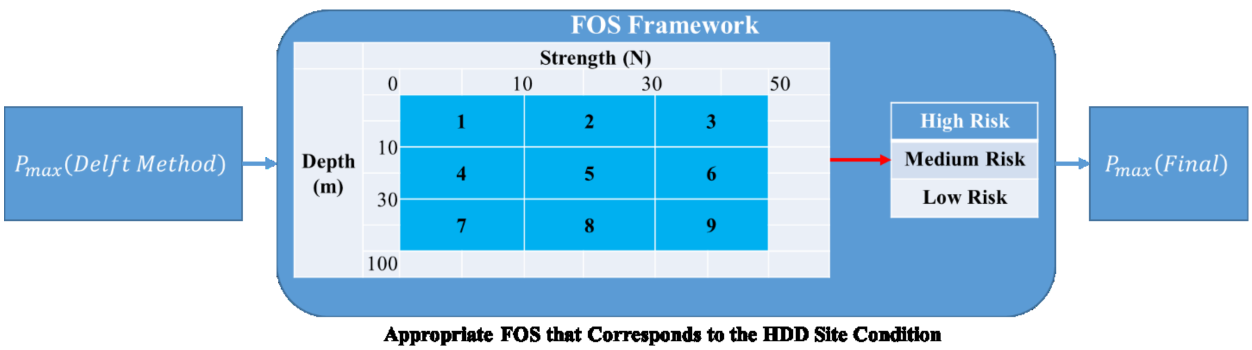

Figure 4.1. Proposed Framework of Factor of Safety

4.2.2.1. First Component of FOS Framework: Correction Factors of $P_{max(Original\ Delft)}$

For the first component of FOS framework, bore depth and soil strength were chosen as the risk variables as presented in Figure 4.1, since the most input parameters of Delft method are dependent on them. According to the SPT-based method recently introduced by Park and Bayat (2020), the input parameters of the Delft method: σ'_0, u, $R_{p,max}$, ϕ, c, and G, are dependent variables of H and N-value. By adopting the SPT-based method, the geotechnical parameters of Delft method – c, ϕ, and G – can be expressed in terms of N-value, and the other parameters – σ'_0, u, and $R_{p,max}$ – can be expressed in terms of H; therefore, H and N-value were set up as the risk variables of the first component of FOS framework. Since R_0 is an independent term of either H or N-value, it was supposed to be considered as another risk variable; however, as previously mentioned in Chapter 3, R_0 is an insensitive parameter of P_{max} for typical HDD projects; thus, R_0 was not selected as a risk variable of the first component of FOS framework. In order to present the multiple conditions of HDD bores with the first component of FOS framework, which are distinguished by the differences between the level of hydrofracture risks, determination of meaningful intervals of the risk variables H and N-value, was required. For selection of intervals of N-value, the classification of relative density of Peck et al. (1974) was adopted. Peck et al. (1974) provided a relationship between the qualitative descriptions of relative density of coarse-grained soils and N-value as presented in Table 4.1. From Table 4.1, it can be found 10 and 30 are the borderline N-values those distinguish between loose, medium dense, and dense coarse-grained soils. These borderline N-values: 10 and 30, were adopted for setting up the first component of the FOS framework.

Table 4.1. Penetration Resistance and Soil Properties on Basis of the Standard Penetration Test (Peck et al. 1974)

N-Value (blows/ft or 305mm)	**Relative Density**
0 to 4	Very Loose
4 to 10	Loose
10 to 30	Medium
30 to 50	Dense
> 50	Very Dense

Bore depth, H, is another risk variable that was required to be set up with appropriate intervals. Unlike the risk classification regarding N-value, which could be set up using the correlation of Peck et al. (1974), risk classification regarding bore depth could not be found during a literature review; hence, the selection of intervals for H had to be done in a subjective manner. The writer selected 10 m and 30 m as borderline H values for categorization of HDD bores as shallow, intermediate, and deep. HDD bores with H less than 10 m were considered to be shallow, and due to the lack of confinement, the level of hydrofracture risk was expected to be high. In contrast, HDD bores with H greater than 30 m were considered to be deep, because the pressure applied from surcharging soil is sufficient to provide good confinement, and the level of hydrofracture risk was expected to be low. By adapting the selected borderline values for N-value and H (10 and 30 for N-value, and 10 m and 30 m for H), the first component of the FOS framework could be divided into nine zones, as presented in Figure 4.2. Since all nine zones were assigned different bore conditions in terms of N-value and H, the associated hydrofracture risks and correction factors for each zone were expected to vary as well. For determination of correction factors for the nine zones in the first component of the FOS framework, ratios between the P_{max} of the original Delft method and the measurement of hydrofracture pressure were required, as presented in Equation [4.2].

$$\text{Correction Factor} = \frac{P_{max\,(Original\,Delft)}}{P_{Hydrofracture}} \qquad [4.2]$$

where: $P_{max(Original\ Delft)}$ is the maximum allowable mud pressure estimated with the original Delft method ($R_{p,max} = H$), and $P_{Hydrofracture}$ is the measurement of hydrofracture pressure.

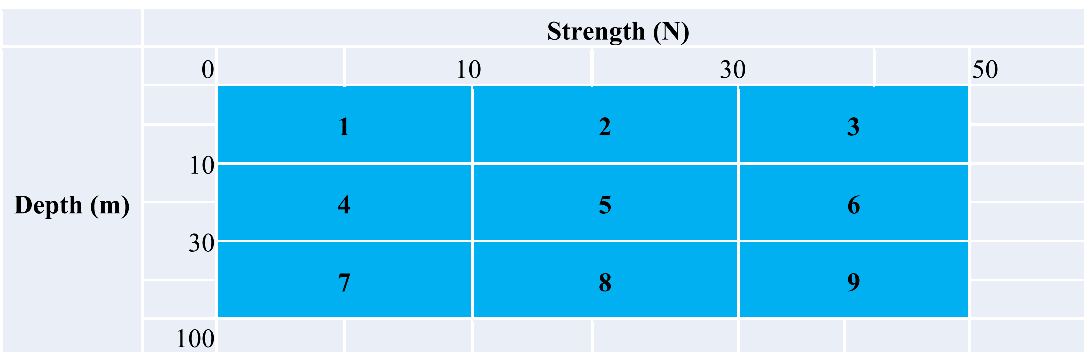

Figure 4.2. First Component of FOS Framework

For determination of correction factors of the first component of FOS framework, data collection of $P_{Hydrofracture}$ was required, which was found to be challenging. There were some literature reports (Luger and Hergarden, 1988; Keulen, 2001; Xia, 2009; Staheli et al., 2010; and Lan and Moore, 2018) available including the measurements of field/laboratory hydrofracture pressure; however, the amount of data was not sufficient to take all nine risk zones with various bore conditions into account (most of the data was from laboratory experiments with very shallow cover and low hydrofracture pressure). Therefore, instead of searching for the measurements of hydrofracture pressure from literature reports (at least for now), the application of currently available P_{max} estimation models was considered, which have been approved to be conservative by majority of practitioners in HDD industry. By combining these approved models together, a conservative P_{max} envelope was expected to be created. For this thesis, three commonly used Delft-based models were selected: a) factored Delft Method with $R_{p,max}$ suggested by Delft Geotechnics (1997),

b) factored Delft Method with $R_{p,max}$ suggested by Staheli et al. (2010), and c) NEN 3650 Method (NEN, 2017). More detailed information about these three Delft-based models is included in the following sections.

4.2.2.1.1. Factored Delft Method with $R_{p,max}$ suggested by Delft Geotechnics (1997)

$$P_{max} = \frac{u + \left[\sigma'_0 (1+\sin\varphi) + c\cos\varphi + c\cot\varphi\right] \times \left[\left(\frac{R_0}{R_{p,max}}\right)^2 + \frac{\sigma'_0 \sin\varphi + c\cos\varphi}{G}\right]^{\frac{-\sin\varphi}{1+\sin\varphi}} - c\cot\varphi}{FOS} \tag{4.3}$$

where: $R_{p,max}$ is 2/3 H for sand layers, and $H/2$ for clayey and peat layers.

Equation [4.3] is a factored version of Delft method with substitution of $R_{p,max}$ suggested by Delft Geotechnics (van Brussel and Hergarden, 1997). According to Miller and Robinson (2019), 2.0 is the commonly used FOS; however, some regulatory systems require higher value for more strict control of hydrofracture failure. For example, New Orleans District Engineering Division of USACE requires 3.0 as a minimum FOS (USACE, 2007).

4.2.2.1.2. Factored Delft Method with $R_{p,max}$ suggested by Staheli et al. (2010)

$$P_{max} = \frac{u + \left[\sigma'_0 (1+\sin\varphi) + c\cos\varphi + c\cot\varphi\right] \times \left[\left(\frac{R_0}{R_{p,max}}\right)^2 + \frac{\sigma'_0 \sin\varphi + c\cos\varphi}{G}\right]^{\frac{-\sin\varphi}{1+\sin\varphi}} - c\cot\varphi}{FOS} \tag{4.4}$$

where: $R_{p,max}$ is $2D_0$ to $3D_0$ or less.

When Staheli et al. (2010) brought up the concept about limiting $R_{p,max}$ as a very small value, they suggested two different approaches for the application of FOS to P_{max}. The first approach was limiting the $R_{p,max}$ to be less than $2D_0$ to $3D_0$, and the second approach was applying an additional FOS to the P_{max} estimated with $R_{p,max}$ as H. Since the definition of $R_{p,max}$ of "less than $2D_0$ to $3D_0$" is back-calculated one using measurements of hydrofracture pressure, P_{max} estimated with such $R_{p,max}$ may not spare any safety margin. In order to ensure some margin of safety for the estimation of P_{max} with $2D_0$ to $3D_0$ as $R_{p,max}$, Miller and Robinson (2019) suggested applying 1.5 as an additional FOS.

4.2.2.1.3. NEN 3650 Method (NEN, 2017)

$$P_{max} = \left(P'_f + c_f \cot \varphi_f\right)\left[\left(\frac{R_0}{R_{p,max}}\right)^2 + Q\right]^{\frac{-\sin\varphi_f}{1+\sin\varphi_f}} - c \cot \varphi_f + u \qquad [4.5]$$

where:

$P'_f =$ $\sigma'_{0f} \times (1 + \sin \varphi_f) + c_f \times \cos \varphi_f$

$Q =$ $\frac{\sigma'_{0f} \times \sin\varphi_f + c_f \times \cos\varphi_f}{G_f}$

$\sigma'_{0f} =$ $\frac{\sigma'_0}{f_\gamma}$ (NEN 3650)

$\sigma'_{0f} =$ $\frac{3}{4}\frac{\sigma'_v}{f_\gamma}$ (D-Geo Pipeline Manual, which is based on NEN 3650)

NOTE: To determine the effective stress the effective weights of the actual soil layers obtained from the geotechnical survey should be assumed. For the other soil parameters, a conservative assumption should be made or it should be demonstrated how a representative parameter set is determined.

$\varphi_f =$ $\tan^{-1}\left(\frac{\tan\varphi}{f_\varphi}\right)$

$G_f =$ $\frac{E}{f_E \times 2(1+\nu)}$

$c_f =$ $\frac{c}{f_c}$

$R_{p,max} =$ $0.5H$ (For Clay or Loam)

$R_{p,max} =$ Smallest value between following: (For Sand)

$R_{p,max} = 0.5H$,

Or

$R_{p,max} = \sqrt{\frac{R_0^2}{Q} \times 2\varepsilon_{g,max}}$, where: $\varepsilon_{g,max}$ for sand, 0.05 can be adopted.

Partial factors are specified in NEN 3650 for each parameter:

$f_\gamma =$ 1.10

$f_\varphi =$ 1.10

$f_E =$ 1.25

$f_c =$ 1.40

4.2.2.1.4. Comparison between the Cavity Expansion Model of Yu and Houlsby (1991) and Delft-based Models

Cavity expansion model for dilative soil proposed by Yu and Houlsby (1991) includes behavior of plastic soil and large strain theory; therefore, this model was considered to be a valuable one for development of realistic P_{max} envelope. However, making direct comparison between the model of Yu and Houlsby (1991) and the other three Delft-based models was found challenging for several reasons. First of all, the model of Yu and Houlsby (1991) includes a series expansion term, which results coding of the model into spreadsheet become more complicated than the other Delft-based models. Secondly, determination of dilation angle, ψ, without a laboratory experiment: direct shear test, triaxial test, etc., is challenging. Lastly, unlike the other Delft-based models, model of Yu and Houlsby (1991) estimates P_{max} requires strain level of cavity; thus, it requires additional procedure of making selection of strain level of bore. Considering all these challenges, instead of directly including the model of Yu and Houlsby (1991) for development of P_{max} envelope, comparisons with the other three Delft-based models under few different setups of H, and N-value, were made to observe how close the estimations of P_{max} and $R_{p,max}$ of each method are. Input parameters applied for the comparisons are presented in Table 4.2, and the rest of soil strength parameters: c, ϕ and G, were obtained by the SPT-based method (Park and Bayat, 2020). Apparent cohesion and dilation angle were briefly assumed for brevity.

Table 4.2. Input Parameters of Model of Yu and Houlsby (1991) Used for Comparison

Mode Coefficient (Cylindrical = 1, and Spherical = 2)	m	1		
Depth of Groundwater Table	D_w	0 m		
Cover Depth	H	10 m	30 m	100 m
Initial Bore Radius,	R_0	0.0762 m (3 inches)		
Soil Density (Wet)	ρ_{sand}	2350 kg/m^3		
SPT N-value	N	10	30	50
Apparent Cohesion	c'	0 kPa	5 kPa	10 kPa
Dilation Angle	ψ	0°	15°	30°

Comparisons between the four models were made for bore depths of 10 m, 30 m, and 100 m. Furthermore, for each depth, soil conditions were also varied as loose ($N = 10$, $c' = 0$ kPa, and $\psi = 0°$), dense ($N = 30$, $c' = 5$ kPa, and $\psi = 15°$), and very dense ($N = 50$, $c' = 10$ kPa, and $\psi = 30°$). For estimation of P_{max} with model of Yu and Houlsby (1991), tangential strain at hydrofracture was required. As it was mentioned in Chapter 2, Keulen suggested 5% as the maximum allowable strain in tangential direction at hydrofracture, and this 5% was adopted as $\varepsilon_{g,max}$ for the estimation of $R_{p,max}$ of NEN 3650 method. Therefore, estimations of P_{max} with model of Yu and Houlsby (1991) were also made with the assumption of tangential strain as 5%. Results of the comparisons of P_{max} and $R_{p,max}$, were presented in Table 4.3 for 10 m depth, Table 4.4 for 30 m depth, and Table 4.5 for 100 m depth. Compared to the other two Delft-based models, the NEN 3650 method resulted in the closest estimations of both P_{max} and $R_{p,max}$ with the model of Yu and Houlsby (1991).

Table 4.3. Comparison Result at 10m Bore Depth

***H* = 10 m**					
Loose (N = 10, ψ = 0°)		Dense (N = 30, ψ = 15°)		Very dense (N = 50, ψ = 30°)	
P_{max}					
$P_{max(Factored\ Delft)}$	487 kPa	$P_{max(Factored\ Delft)}$	842 kPa	$P_{max(Factored\ Delft)}$	1032 kPa
$P_{max(Factored\ Staheli)}$	396 kPa	$P_{max(Factored\ Staheli)}$	495 kPa	$P_{max(Factored\ Staheli)}$	540 kPa
$P_{max(NEN\ 3650)}$	388 kPa	$P_{max(NEN\ 3650)}$	540 kPa	$P_{max(NEN\ 3650)}$	621 kPa
$P_{max(Yu\ \&\ Houlsby,\ 1991)}$	373 kPa	$P_{max(Yu\ \&\ Houlsby,\ 1991)}$	638 kPa	$P_{max(Yu\ \&\ Houlsby,\ 1991)}$	877 kPa
$R_{p,max}$					
$R_{p,max\ (Factored\ Delft)}$	6.667 m	$R_{p,max\ (Factored\ Delft)}$	6.667 m	$R_{p,max\ (Factored\ Delft)}$	6.667 m
$R_{p,max\ (Factored\ Staheli)}$	0.305 m	$R_{p,max\ (Factored\ Staheli)}$	0.305 m	$R_{p,max\ (Factored\ Staheli)}$	0.305 m
$R_{p,max\ (NEN\ 3650)}$	0.236 m	$R_{p,max\ (NEN\ 3650)}$	0.308 m	$R_{p,max\ (NEN\ 3650)}$	0.336 m
$R_{p,max\ (Yu\ \&\ Houlsby,\ 1991)}$	0.197 m	$R_{p,max\ (Yu\ \&\ Houlsby,\ 1991)}$	0.291 m	$R_{p,max\ (Yu\ \&\ Houlsby,\ 1991)}$	0.369 m

Table 4.4. Comparison Result at 30m Bore Depth

***H* = 30 m**					
Loose (N = 10, ψ = 0°)		Dense (N = 30, ψ = 15°)		Very dense (N = 50, ψ = 30°)	
P_{max}					
$P_{max(Factored\ Delft)}$	949 kPa	$P_{max(Factored\ Delft)}$	1451 kPa	$P_{max(Factored\ Delft)}$	1749 kPa
$P_{max(Factored\ Staheli)}$	959 kPa	$P_{max(Factored\ Staheli)}$	1214 kPa	$P_{max(Factored\ Staheli)}$	1345 kPa
$P_{max(NEN\ 3650)}$	860 kPa	$P_{max(NEN\ 3650)}$	1095 kPa	$P_{max(NEN\ 3650)}$	1221 kPa
$P_{max(Yu\ \&\ Houlsby,\ 1991)}$	761 kPa	$P_{max(Yu\ \&\ Houlsby,\ 1991)}$	1115 kPa	$P_{max(Yu\ \&\ Houlsby,\ 1991)}$	1387 kPa
$R_{p,max}$					
$R_{p,max\ (Factored\ Delft)}$	20.000 m	$R_{p,max\ (Factored\ Delft)}$	20.000 m	$R_{p,max\ (Factored\ Delft)}$	20.000 m
$R_{p,max\ (Factored\ Staheli)}$	0.305 m	$R_{p,max\ (Factored\ Staheli)}$	0.305 m	$R_{p,max\ (Factored\ Staheli)}$	0.305 m
$R_{p,max\ (NEN\ 3650)}$	0.151 m	$R_{p,max\ (NEN\ 3650)}$	0.193 m	$R_{p,max\ (NEN\ 3650)}$	0.208 m
$R_{p,max\ (Yu\ \&\ Houlsby,\ 1991)}$	0.134 m	$R_{p,max\ (Yu\ \&\ Houlsby,\ 1991)}$	0.181 m	$R_{p,max\ (Yu\ \&\ Houlsby,\ 1991)}$	0.213 m

Table 4.5. Comparison Result at 100m Bore Depth

***H* = 100 m**					
Loose (N = 10, ψ = 0°)		Dense (N = 30, ψ = 15°)		Very dense (N = 50, ψ = 30°)	
P_{max}					
$P_{max(Factored\ Delft)}$	2090 kPa	$P_{max(Factored\ Delft)}$	2837 kPa	$P_{max(Factored\ Delft)}$	3287 kPa
$P_{max(Factored\ Staheli)}$	2494 kPa	$P_{max(Factored\ Staheli)}$	3076 kPa	$P_{max(Factored\ Staheli)}$	3399 kPa
$P_{max(NEN\ 3650)}$	2279 kPa	$P_{max(NEN\ 3650)}$	2638 kPa	$P_{max(NEN\ 3650)}$	2831 kPa
$P_{max(Yu\ \&\ Houlsby,\ 1991)}$	1832 kPa	$P_{max(Yu\ \&\ Houlsby,\ 1991)}$	2315 kPa	$P_{max(Yu\ \&\ Houlsby,\ 1991)}$	2621 kPa
$R_{p,max}$					
$R_{p,max\ (Factored\ Delft)}$	66.667 m	$R_{p,max\ (Factored\ Delft)}$	66.667 m	$R_{p,max\ (Factored\ Delft)}$	66.667 m
$R_{p,max\ (Factored\ Staheli)}$	0.305 m	$R_{p,max\ (Factored\ Staheli)}$	0.305 m	$R_{p,max\ (Factored\ Staheli)}$	0.305 m
$R_{p,max\ (NEN\ 3650)}$	0.097 m	$R_{p,max\ (NEN\ 3650)}$	0.121 m	$R_{p,max\ (NEN\ 3650)}$	0.129 m
$R_{p,max\ (Yu\ \&\ Houlsby,\ 1991)}$	0.092 m	$R_{p,max\ (Yu\ \&\ Houlsby,\ 1991)}$	0.114 m	$R_{p,max\ (Yu\ \&\ Houlsby,\ 1991)}$	0.125 m

4.2.2.1.5. Comparison of Three Factored Delft-Based Models

For comparison between the three factored Delft-based models: the factored Delft method with $R_{p,max}$ suggested by Delft Geotechnics (1997), factored Delft method with $R_{p,max}$ suggested by Staheli et al. (2010), and NEN 3650 method (NEN, 2017), each was plotted together in P_{max} vs. N format. The SPT-based method was adopted for conversion of N-value into geotechnical parameters; c, ϕ, and G. The in-situ condition of Table 4.2, which was used for the simulation of model of Yu and Houlsby (1991), was applied for the comparison between the three Delft-based models. Comparisons are presented with three different bore depths: 10 m for Figure 4.3, 30 m for Figure 4.4, and 100 m for Figure 4.5. In these figures, the factored Delft method with $R_{p,max}$ suggested by Delft Geotechnics (1997) is labeled as "($R_{p,max}$ = 2/3 H, FOS = 2.0)", and factored Delft method with $R_{p,max}$ suggested by Staheli et al. (2010) is labeled as "($R_{p,max}$ = 2D_0 to 3D_0, FOS = 1.5)".

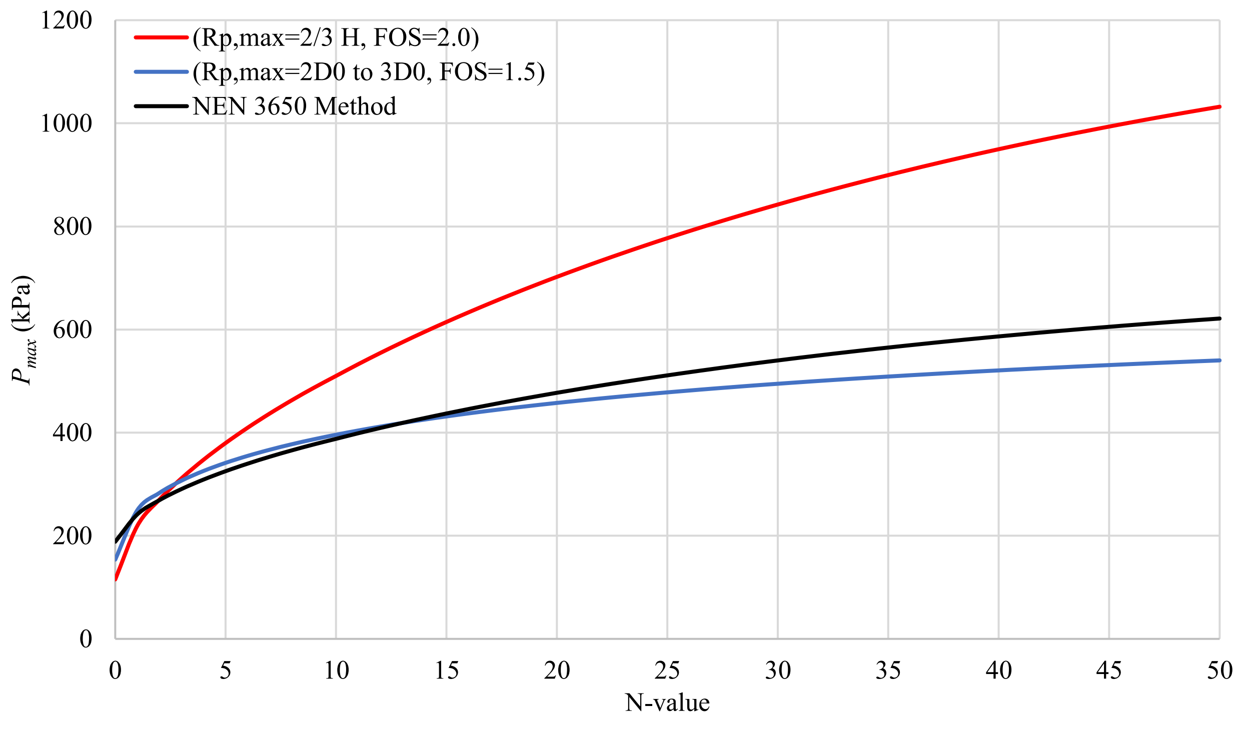

Figure 4.3. Comparison of Three Models at 10 m Bore Depth

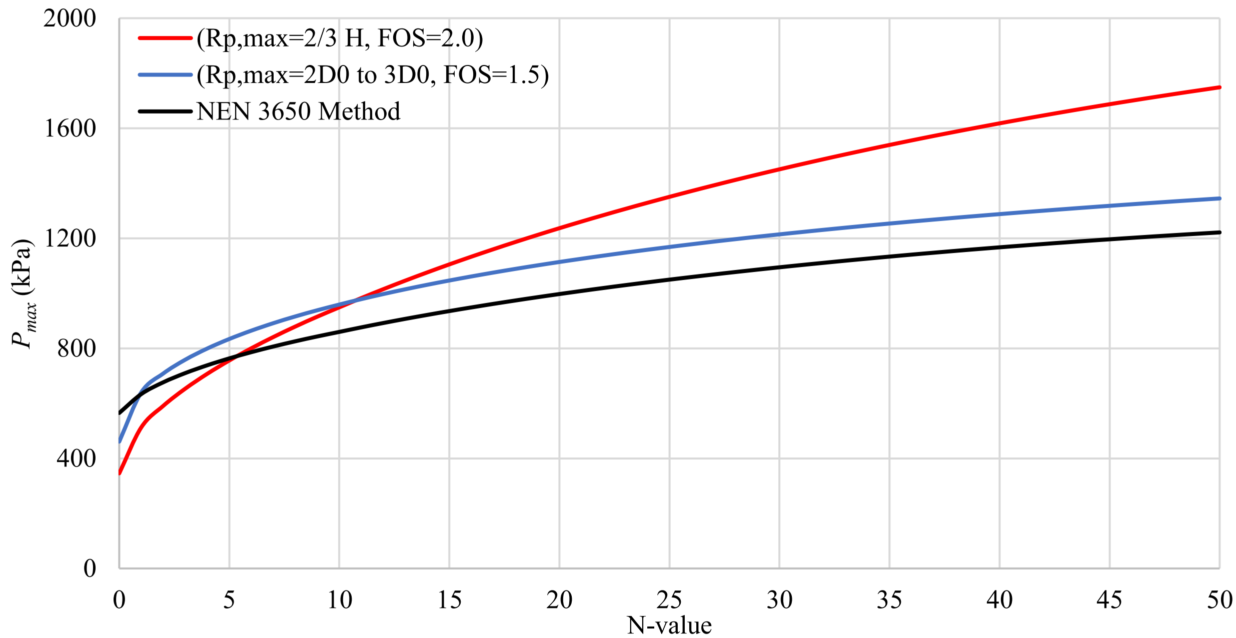

Figure 4.4. Comparison of Three Models at 30 m Bore Depth

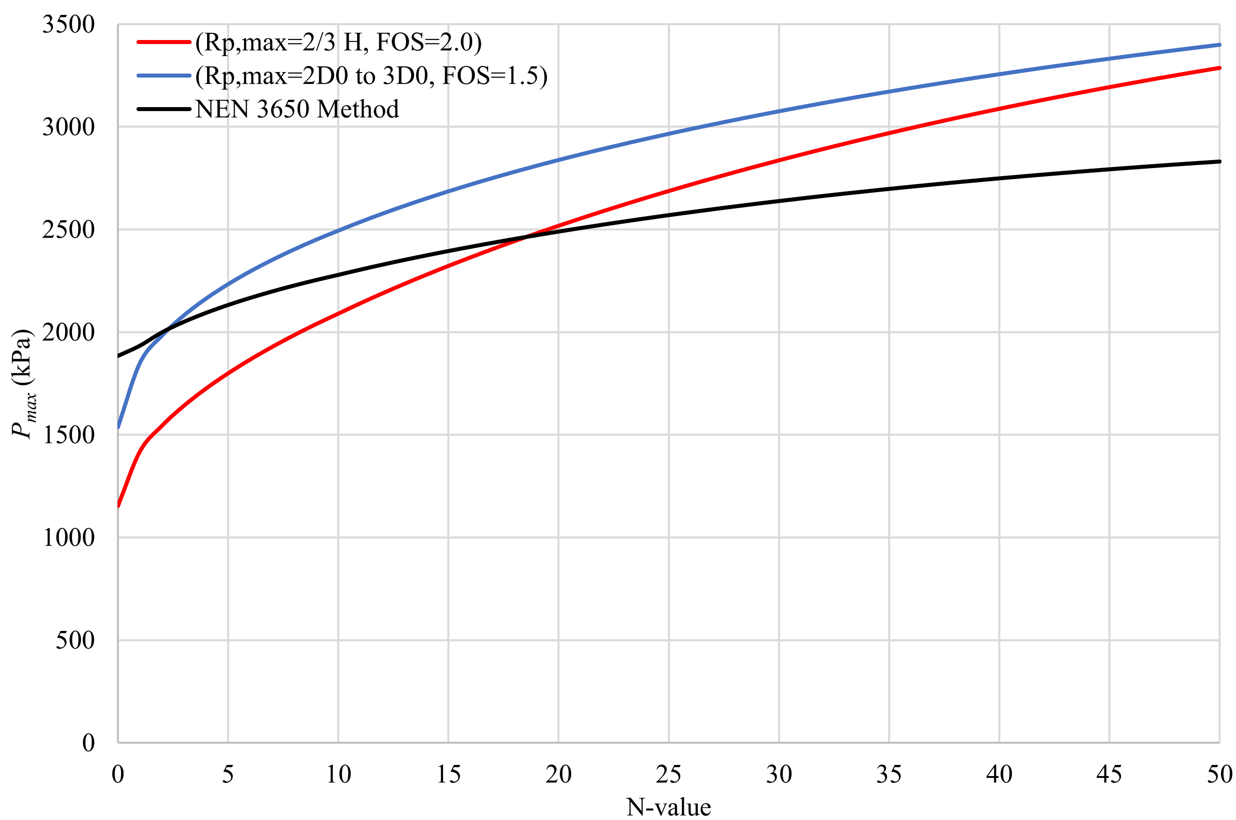

Figure 4.5. Comparison of Three Models at 100 m Bore Depth

From the comparisons of Figure 4.3, 4.4, and 4.5, it can be observed that none of the three Delft-based models is absolutely more conservative than the other ones for all different conditions. For brevity, the factored Delft method with $R_{p,max}$ suggested by Delft Geotechnics (1997) is mentioned as "($R_{p,max}$ = 2/3 H, FOS = 2.0)", and the factored Delft method with $R_{p,max}$ suggested by Staheli et al. (2010) is mentioned as "($R_{p,max}$ = 2D_0 to 3D_0, FOS = 1.5)".

For a 10 m depth bore, which is presented in Figure 4.3 as a simulation of shallow bore, the most conservative estimations of P_{max} are made by "($R_{p,max}$ = 2/3 H, FOS = 2.0)" for $N \leq 2$, NEN 3650 method for $2 \leq N \leq 13$, and "($R_{p,max}$ = 2D_0 to 3D_0, FOS = 1.5)" for $N \geq 13$. Compared to the estimations of P_{max} of three Delft-based models staying in a close range until $N \leq 3$, "($R_{p,max}$ = 2/3 H, FOS = 2.0)" begins to deviate from the other two models and it becomes almost twice at N = 50. "($R_{p,max}$ = 2D_0 to 3D_0, FOS = 1.5)" and NEN 3650 method show good agreement over the entire range of N-value, and their difference in the estimations of P_{max} is kept within 100kPa.

For a 30 m depth bore, which is presented in Figure 4.4 as a simulation of intermediate bore, the most conservative estimations of P_{max} are made by "($R_{p,max}$ = 2/3 H, FOS = 2.0)" for $N \leq 5$, and NEN 3650 method for $5 \leq N$. "($R_{p,max}$ = 2/3 H, FOS = 2.0)" begins to deviate from the other two models at $N \leq 10$, and it becomes almost 1.5 times of the NEN 3650 method at N = 50. "($R_{p,max}$ = 2D_0 to 3D_0, FOS = 1.5)" and NEN 3650 method show relatively good agreement over the entire range of N-value, and their difference in P_{max} predictions is kept within 150kPa.

For a 100 m depth bore, which is presented in Figure 4.5 as a simulation of deep bore, the most conservative estimations of P_{max} are made by "($R_{p,max}$ = 2/3 H, FOS = 2.0)" for $N \leq 19$, and NEN 3650 method for $N \geq 19$. Estimations of P_{max} of "($R_{p,max}$ = 2/3 H, FOS = 2.0)" and "($R_{p,max}$ = 2D_0 to 3D_0, FOS = 1.5)" does not have good agreement with NEN 3650 method over the entire range of N-value, and their differences become greater than 400kPa at N = 50.

4.2.2.1.6. Finding the Correction Factors for the First Component of FOS Framework with Delft-based Models

According to Equation [4.2], correction factors of the first component of FOS framework can be obtained from the ratios of $P_{max(Original\ Delft)}$ and $P_{Hydrofracture}$. For this thesis, it was decided to use the P_{max} estimations of Delft-based models, instead of measurements of hydrofracture pressure; therefore, Equation [4.2] could be modified as presented in Equation [4.5].

$$\text{Correction Factor} = \frac{P_{max\,(Original\,Delft)}}{P_{Hydrofracture}} = \frac{P_{max\,(Original\,Delft)}}{P_{max\,(Delft-based\,Model)}} \tag{4.5}$$

After calculating all the correction factors using different combinations of H and N-values, it was required to find the representative values for each risk zone of the first component of FOS framework. For selection of the representative values for risk zones, either maximum, or average could be considered as options. The correction factor selected using the maximum value of each zone may result excessively conservative estimation of P_{max} for the most cases; moreover, it cannot reflect the overall trend of the entire correction factor data. Since the correction factor selected with average value of each zone was considered to be the better representation of the entire data, average values were chosen for determination of correction factors for the first component of FOS framework.

4.2.2.1.7. Ineffectiveness of the $R_{p,max}$ suggested by Delft Geotechnics (1997) Presented with FOS Framework

As previously mentioned, $R_{p,max}$ suggested by Delft Geotechnics (1997) is known to be ineffective on P_{max}. Using the correction factor of the first component of FOS framework, such ineffectiveness of $R_{p,max}$ suggestion by Delft Geotechnics (1997) could be conveniently explained. Equation [4.5] of correction factors of the first component of FOS framework could be applied as presented in Equation [4.6].

$$\text{Correction Factor} = \frac{P_{max\,(Original\,Delft)}}{P_{max\,(Delft-based\,Model)}} = \frac{P_{max\,(Original\,Delft)}}{P_{max}\left(\frac{2}{3}H\right)} = \frac{P_{max}(H)}{P_{max}\left(\frac{2}{3}H\right)} \tag{4.6}$$

By substituting the input parameters of Table 4.2 into Equation [4.6], correction factors could be obtained as presented in Figure 4.6. Estimations of P_{max} with $R_{p,max}$ suggested by Delft Geotechnics (1997) at shallow depth (less than 10m) were reduced up to 5%; however, the ones deeper than 10m did not show any difference. Considering such insensivity from this result, $R_{p,max}$ suggested by Delft Geotechnics (1997) does not seem to secure sufficient margin of safety.

	Strength (N)		
Depth (m)	**0–10**	**10–30**	**30–50**
0–10	**1.02**	**1.04**	**1.05**
10–30	**1.00**	**1.00**	**1.00**
30–100	**1.00**	**1.00**	**1.00**

Figure 4.6. Effect of the $R_{p,max}$ Suggested by Delft Geotechnics (1997)

4.2.2.1.8. Comparison between the Original Delft Method and the Three Delft-Based Models

For comparison between the original Delft method and the factored Delft method with $R_{p,max}$ suggested by Delft Geotechnics (1997), Equation [4.5] was modified as presented in Equation [4.7].

$$\text{Correction Factor} = \frac{P_{max\,(Original\,Delft)}}{P_{max\,(Delft-based\,Model)}} = P_{max}(H) \Bigg/ \frac{P_{max}\left(\frac{2}{3}H\right)}{FOS=2.0} \qquad [4.7]$$

By substituting the input parameters of Table 4.2 into Equation [4.7] and selecting the average values of correction factors for each risk zone, the first component of FOS framework could be obtained as presented in Figure 4.7. Correction factor values presented in Figure 4.7 were basically twice of the ones presented in Figure 4.6 and they were consistent through all the nine zones.

	Strength (N)		
Depth (m)	**0–10**	**10–30**	**30–50**
0–10	**2.03**	**2.08**	**2.11**
10–30	**2.00**	**2.00**	**2.00**
30–100	**2.00**	**2.00**	**2.00**

Figure 4.7. First Component of FOS Framework based on the Comparison with Factored Delft Method with $R_{p,max}$ Suggested by Delft Geotechnics (1997)

For comparison between the original Delft method and the factored Delft method with $R_{p,max}$ suggested by Staheli et al. (2010), Equation [4.5] could be modified as presented in Equation [4.8].

$$\text{Correction Factor} = \frac{P_{max\,(Original\,Delft)}}{P_{max\,(Delft-based\,Model)}} = P_{max}(H) \Big/ \frac{P_{max}(2D_0)}{FOS=1.5} \qquad [4.8]$$

By substituting the input parameters of Table 4.2 into Equation [4.8] and selecting the average values of correction factors for each risk zone, the first component of FOS framework could be obtained as presented in Figure 4.8. Unlike the factored Delft method with $R_{p,max}$ suggested by Delft Geotechnics (1997), the factored Delft method with $R_{p,max}$ suggested by Staheli et al. (2010) resulted more diverse correction factors. From the factored Delft method with $R_{p,max}$ suggested by Staheli et al. (2010), while zone 7 with large depth and low soil strength presented the smallest correction factor of 1.66, zone 3 with shallow depth and high soil strength presented the largest correction factor of 4.53. Correction factors tended to be larger as the H is smaller, and N-value is greater.

Depth (m) \ Strength (N)	0–10	10–30	30–50
0–10	**2.52**	**3.77**	**4.53**
10–30	**1.91**	**2.50**	**2.90**
30–100	**1.66**	**1.92**	**2.10**

Figure 4.8. First Component of FOS Framework based on the Comparison with Factored Delft method with $R_{p,max}$ Suggested by Staheli et al. (2010)

For comparison between the original Delft method and NEN 3650 method, Equation [4.5] could be modified as presented in Equation [4.9].

$$\text{Correction Factor} = \frac{P_{max\,(Original\,Delft)}}{P_{max\,(Delft-based\,Model)}} = P_{max}(H) \Big/ P_{max\,(NEN\,3650\,Method)} \qquad [4.9]$$

By substituting the input parameters of Table 4.2 into Equation [4.9] and taking the average values of correction factors for each risk zone, the first component of FOS framework could be obtained as presented in Figure 4.9. Similar to the factored Delft method with $R_{p,max}$ suggested by Staheli et al. (2010), NEN 3650 method also presented diverse correction factors over the risk zones. However, deviation between the correction factors of NEN 3650 method was smaller than the one of factored Delft method with $R_{p,max}$ suggested by Staheli et al. (2010).

	Strength (N)		
Depth (m)	0–10	10–30	30–50
0–10	**2.37**	**3.05**	**3.29**
10–30	**2.02**	**2.66**	**2.96**
30–100	**1.74**	**2.19**	**2.45**

Figure 4.9. First Component of FOS Framework based on the Comparison with NEN 3650 Method

4.2.2.1.9. Combined FOS Framework of All Three Delft-based Models

After all the correction factors were obtained from the comparison between the original Delft method and the three Delft-based models, they were combined together into a single set of correction factors. In this thesis, combination of the three sets of correction factors in Figure 4.7, 4.8, and 4.9 was simply made by taking average values. The resulting set of correction factors was presented in Figure 4.10.

	Strength (N)		
Depth (m)	0–10	10–30	30–50
0–10	**2.31**	**2.97**	**3.31**
10–30	**1.98**	**2.39**	**2.62**
30–100	**1.80**	**2.04**	**2.18**

Figure 4.10. Combined Correction Factors for the First Component of FOS Framework

4.2.2.2. Second Component of FOS Framework: Risk-based Factor

In geotechnical engineering, probabilistic methods have been extensively applied for designs. For determination of partial factors of NEN 3650 method: f_γ, f_ϕ, f_E, and f_c, probabilistic methods were also used to take the uncertainty of input geotechnical parameters: γ, ϕ, E, and c, into account (Guijt et al., 2004). However, the Geo-Institute of ASCE recently recognized the consequence of a potential failure also needs to be considered in design decision (Gross, 2019). Following the trend toward risk-informed design, the second component of FOS framework of this thesis was proposed to be risk-based as well. The consequence of potential failure of the HDD operation due to the excessive drilling fluid pressure would be hydrofracture. If hydrofracture occurs during the HDD operation and drilling fluid escapes into surrounding environment, remediation procedures must be taken, which generally requires additional cost. This remediation cost may vary depending on the type of surrounding environment. For example, remediation costs required for river-crossing project and urban area-crossing project are not likely to be the same. Moreover, repair costs for ground heaves on railways and local roads might have significant differences. If the environment types of HDD application are categorized by the level of remediation costs and hydrofracture risks, they can be adapted for development of risk-based factors of the second component of FOS framework. Once the second component of FOS framework, $\text{FOS}_{\text{Risk-based}}$, is determined, it can be multiplied to the correction factor of the first component of FOS framework to obtain the finalized FOS as presented in Equation [4.10].

$$\text{FOS} = \text{Correction Factor} \times \text{FOS}_{\text{Risk-based}} \qquad [4.10]$$

With application of the FOS of Equation [4.10], the finalized maximum allowable mud pressure, $P_{max(Final)}$, can be obtained as presented in Equation [4.11].

$$P_{max(Final)} = \frac{P_{max\,(Original\ Delft)}}{\text{FOS}} \qquad [4.11]$$

Determination of risk-based factors for the second component of FOS framework may require design and cost information from real world projects. For brevity, completion of the second component of FOS framework was not included in this thesis; however, it will be discussed in future research with risk assessments and cost analyses of the actual hydrofracture cases.

4.3. VALIDATIONS

For validation of the FOS framework proposed in Chapter 4, information from two hydrofracture case studies those were provided by Keulen (2001), and Staheli et al. (2010), were used. Staheli et al. (2010) provided their own field measurement data from two sites, and Keulen (2001) referenced to the information from another literature, which is referred as "BTL 48 – Blow out Experiment". For the estimation of P_{max} of original Delft method, SPT-base method (Park and Bayat, 2020) was adopted.

4.3.1. Case 1: Staheli et al. (2010) – Location 2

Staheli et al. (2010) included two field measurement data of hydrofracture with two different type of soils. Since the FOS framework proposed in this thesis is only limited to coarse-grained soils, case study of location 1, which involves fine-grained soil could not be used. Case study of location 2 involved cohesionless soil; therefore, it could be used for the validation. Soil found from location 2 was "medium dense sands with some silts and gravel". The site geometry and soil parameters provided are presented in Table 4.6.

Table 4.6. Location 2 – Geometry and Soil Parameters (Staheli et al., 2010)

Depth of the Bore below Ground Surface	h_s	30 ft	10 m
Height of Groundwater over the Bore	h_w	30 ft	10 m
Bore Radius	R_0	6 in	6 in
Unit Weight of Soil above the Groundwater	γ	130 pcf	20.42 kN/m^3
Soil Friction Angle	ϕ	28°	28°
Cohesion	c	0	0
Poisson's Ratio	v	0.30	0.30
Modulus of Elasticity	E	125 tsf	11970 kPa
Shear Modulus	G	96000 psf	4597 kPa

Since this case study was already presented in Chapter 3 for validation of SPT-based method, input parameters of Delft method: σ'_0, u, and N-value, are known as 106.1 kPa, 98 kPa, and 8. With the substitution of 8 as N-value into SPT-based method (Park and Bayat, 2020), its input geotechnical parameters: φ, and G, are estimated as 30° and 4378 kPa. For determination of FOS using the newly proposed framework of this chapter, correction factor and risk-based factor are required. Since the risk variables of the first component of FOS framework: N-value and H, are 8 and 10m, correction factor of 2.31 can be obtained from the Zone 1 of Figure 4.10. With an assumption of risk-based factor as

1, FOS of this HDD bore could be determined as 2.31. By substituting 106.1 kPa, 98 kPa, 30°, 4378 kPa, and 2.31 into σ'_0, u, φ, G, and FOS of the SPT-based method (Park and Bayat, 2020), $P_{max(Final)}$ can be estimated as 344 kPa. According to Staheli et al. (2010), the measurement of hydrofracture pressure at Location 2 was 55 psi, which is equivalent to 379 kPa. The estimation of $P_{max(Final)}$ using the FOS framework is slightly less than the measurement of hydrofracture pressure; however, they are still reasonably close to each other.

4.3.2. Case 2: Keulen (2001) – BTL 48 – Blow out Experiment

BTL stands for "Boren van Tunnels en Leidingen", which means "Drilling of Tunnels and Pipes" in Dutch. This literature was introduced by Keulen (2001) in her master's thesis, and it included information about the scale tests with a cylindrical cavity in sand were performed in GeoDelft Laboratory. Instead of having the actual surcharging soil on top of cavity, a pressure plate was used to simulate the in-situ confinement. The test proceeded until the surrounding sand failed due to fracturing. The default values assumed by BLT 48 are presented in Table 4.7.

Table 4.7. Default Values BTL48 (Keulen, 2001)

Default Values		Value	Unit
Diameter Borehole	D_0	30	mm
Angle of Internal Friction	φ	40	°
Poisson's Ratio	v	0.26	
Elasticity Modulus	E	15000	kPa
Cohesion	c	0	kPa
Initial Effective Stress	σ'_0	160	kPa
Maximum Radius Plastic Zone	$R_{p,max}$	300	mm
Tangential Strain (Assumed)	$\varepsilon_{tt,max}$	2	%
Dilatancy Angle (Assumed)	ψ	0	°

Since the pressure plate was used instead of the actual surcharging soil, H and soil density are not provided from this case study. By assuming the density of sand as 2000 kg/m^3, H can be back-calculated as 8.15m. Considering the given values of σ'_0, φ, and E of 160 kPa, 40°, and 15000 kPa from Table 4.7, pseudo-N-value can be back-calculated as 13. With substitution of 13 as N-value into SPT-based method (Park and Bayat, 2020), its input geotechnical parameters: φ, and G, are estimated as 32° and 15791 kPa (given 40° was considered to be overestimated). By using the risk variables of the first component of FOS framework: N-value and H, are 13 and 8.15m, correction factor of 2.97 can be obtained from the Zone 2 of Figure 4.10. With an assumption of risk-based factor as 1, FOS of this HDD bore could be determined as 2.97. By substituting 160 kPa, 0 kPa, 32°, 15971 kPa, and 2.97 into σ'_0, u, φ, G, and FOS of the SPT-based method (Park and Bayat, 2020), $P_{max(Final)}$can be estimated as 366 kPa. According to Keulen (2001), the measured hydrofracture pressure during the BTL 48 – Blow out experiments were between 370kPa to 400kPa. The estimation of $P_{max(Final)}$ using the FOS framework is slightly less than the measurement of hydrofracture pressure; however, they are still reasonably close to each other.

4.4. CONCLUSION

For improvement of the existing factor of safety concept about the maximum allowable pressure of drilling fluid in HDD, commonly used factored methods were carefully reviewed. Three methods – the factored Delft method with $R_{p,max}$ suggested by Delft Geotechnics (1997), factored Delft method with $R_{p,max}$ suggested by Staheli et al. (2010), and NEN 3650 method (NEN, 2017) – are modified versions of the original Delft method with different approaches for ensuring margin of safety. To observe the degree of conservatism being applied for each method, three models were compared with the original Delft method in various conditions of bore depth and soil strength. As a result, the ratios between the three models and the original Delft method were obtained, and they were combined together by taking average values. From the validation with two case studies from literatures, estimations made with the factor of safety values using the newly proposed framework were found to be close to the measurements of hydrofracture pressure. This factor of safety generated from the comparisons between the models became the first component of FOS framework, which takes the various conditions of bore depth and soil strength into account.

A concept for the second component of FOS framework was proposed to bring attention to the consequence of failure. For the second component, a method of adapting risk-based design factor was suggested. Determination of such risk-based factor requires design and cost information from real projects to quantify the risk. In the future research, the first component of FOS framework should be updated with more pressure measurement data from hydrofracture cases to improve its accuracy of P_{max}. By updating the FOS framework with more data from real HDD projects, it will be able to contribute on advancement of design of the limiting annular pressure with better accuracy, and more suitable choices for variations existing in each project. Furthermore, by adopting the IR assessment method for clays, such as Queen's method (Xia, 2009), estimation of P_{max} with FOS framework may be extended for the fine-grained soils as well.

REFERENCES

Deltares, (2016), – *D-GEO PIPELINE: Design of Pipeline Installation User Manual*, Version 16.1, Deltares, Delft, 2016, 324p

Elwood, D.E.Y., (2008), "Hydraulic Fracture Experiments in a Frictional Material and Approximation for Maximum Allowable Mud Pressure," M.Sc. Thesis, Queens University, Kingston, Ontario, Canada, July 2008

Goerz, S., Boelhouwer, N., and Taylor, J., (2019), "A New Perspective in Hydrofracture Analysis", *Proceedings of 2019 No-Dig Conference*, Chicago, Illinois, March 17-20, 2019

Gross, B.A., (2019), "What's New at the Geo-Institute?" *Geostrata*, July/August, 2019, pp. 8-9.

Guijt, W., Vrouwenvelder, T., Grensnigt, N., and Dijkstra, G., (2004), "Safety Concept in The New Dutch Pipeline Standard NEN 3650", *Proceedings of the Fourth International Offshore and Polar Engineering Conference*, Toulon, France, May 23-28, 2004

Keulen, B., (2001), "Maximum Allowable Pressures During Horizontal Directional Drillings Focused on Sand", Master thesis, Delft University of Technology, Delft, Netherlands.

Lan, H., and Moore, I.D., (2018), "Practical Criteria for Borehole Instability in Sand during Horizontal Directional Drilling," *Proceedings of 2018 No – Dig Conference*, Palm Springs, California, March 20-24, 2018

Luger, H.J., and Hergarden, H.J.A.M., (1988) "Directional Drilling in Soft Soil: Influence of Mud Pressure", International Society of Trenchless Technology, *No-Dig Conference*, Washington, D.C., October 16-19, 1988

Miller, M.A., and Robinson, J.L., (2019), "Soil Mechanics and Calculating Hydraulic Fracture Risk," *Proceedings of 2019 No-Dig Conference*, Chicago, Illinois, March 18-21, 2019

NEN, (2006), NEN 3650-1:2003+A1:2006. Eisen voor buisleidingsystemen - Deel 1: Algemene – Katern 1 tot en met 6 (Requirements for Pipeline Systems - Part 1: General – Quire 1 to 6), in English

NEN, (2017), NEN 3650-1+C1:2017. Eisen voor buisleidingsystemen - Deel 1: Algemene eisen (Requirements for Pipeline Systems - Part 1: General Requirements), in English

Park, I., and Bayat, A., (2020), "Simplified Application of the Delft Method to Estimate Maximum Allowable Annular Pressure in HDD", *Proceedings of 2020 Pipelines Conference*, ASCE, San Antonio, Texas (Conference Held Virtually), August 9-12, 2020

Peck, R.B., Hanson, W.E., and Thorburn, T.H., (1974), – *Foundation Engineering*, 2nd Ed., John Wiley and Sons, New York, 1974, 310p

Rostami, A., (2017), "Prediction and Evaluation of Annular Pressure in Horizontal Directional Drilling," Ph.D. Thesis, University of Alberta, Edmonton, Alberta, Canada, April 2017

Staheli, K., Bennett, D., O'Donnell, H.W., and Hurley, T.J., (1998), "Installation of Pipelines beneath Levees Using Horizontal Directional Drilling", CPAR-GL-98-1, April 1998

Staheli, K., Price, C.G., and Wetter, L., (2010), "Effectiveness of Hydrofracture Prediction for HDD Design," *Proceedings of 2010 No - Dig Conference*, Chicago, Illinois, May 2-7, 2010

USACE, (2007), Hurricane and Storm Damage Reduction System Design Guidelines. Interim. U.S. Army Corps of Engineers, New Orleans District. October 23, 2007.

Verruijt, A., (1993), – *Grondmechanica*, 4th Ed., Delftse Uitgevers Maatschappij, Delft, Netherlands

Van Brussel, G.G., and Hergarden, H.J.A.M., (1997), "Research CPAR program: Installation of Pipelines beneath Levees Using Horizontal Directional Drilling", Dept. of Foundations and Underground Engineering, Delft Geotechnics, Delft, Netherlands, pp. 1-16.

Xia, H., (2009), "Investigation of Maximum Mud Pressure within Sand and Clay during Horizontal Directional Drilling", Ph.D. Thesis, Queens University, Kingston, Ontario, Canada, January 2009

Yu, H.S., and Houlsby, G.T., (1991), "Finite Cavity Expansion in Dilatant Soils: Loading Analysis", *Geotechnique*, 41, No. 2, pp. 173-183.

5. Conclusions and Future Work

In Chapter 2, a review of literature related to HDD, hydrofracture, cavity expansion theories, site investigation methods, and correlations between the geotechnical parameters and N-value, was presented. Information about SPT and its application was explained in detail, particularly for site investigation methods and correlation models. Methods developed using cavity expansion theory were explained in chronological order to present the history of improvement of estimation of P_{max}.

In Chapter 3, SPT-based method and its validation works were presented. The SPT-based method was categorized into drained and undrained soil models, and the assumptions required for each condition were explained. Comparisons between the correlation models for geotechnical parameters were presented; moreover, the reasons behind selections of the correlation models were explained in relation to HDD. The limitations and recommendations for use of the SPT-based method were explained, and its algorithm was presented in flow chart form. For validation, measurements of hydrofracture pressure from multiple literatures were used and the accuracy of estimation of SPT-based method was compared with the ones done by the other researchers.

In Chapter 4, development of the framework of factor of safety for Delft method was presented with validation. The FOS framework was divided into two components, where the first component applies a correction factor to the estimation of the original Delft method, and the second component applies a risk-based factor. The first component of FOS framework was set up with soil strength in terms of N-value (the SPT-based method from Chapter 3 was adopted) and depth of cover as risk variables, and it was divided into nine zones with different bore conditions with distinguished hydrofracture risk levels. Since data related to measurements of hydrofracture pressure was not sufficient to cover all the nine zones, currently available factored Delft-based methods were adopted for the search of the values of correction factors for the first component of FOS framework of each zone. The second component of the FOS framework was proposed to bring a new concept about risk-informed design; i.e., depending on the types of surrounding environment, the consequence of hydrofracture was expected to be varied. Subsequently, risk-based design factor of the second component of FOS framework was expected to vary as well. The proposed FOS framework was also validated using the measurements of hydrofracture pressure from multiple literatures.

The key findings of the research are highlighted below.

(1) A newly developed SPT-based method allows direct application of N-value for the systematic estimation of P_{max}. Depending on the soil type and its drainage condition, either drained or undrained soil models can be selected.

(2) Compared to the undrained soil model which presents a linear relationship between the normalized P_{max} (with respect to initial effective stress) and N-value, the drained soil model presents a non-linear relationship. Compared to the P_{max} estimated with the undrained soil model, P_{max} estimated with drained soil model was found much greater for the same N-value.

(3) Through the validation, it was found the estimation of P_{max} using SPT-based method was found to be more accurate and consistent compared to the ones provided by other researchers (Keulen, 2001; Elwood, 2008; Xia, 2009; and Rostami, 2017). A minimum factor of safety of 2.0 or higher is strongly recommended for the SPT-based method.

(4) By making comparisons between the original Delft method and three commonly used factored Delft-based models, the first component of FOS framework could be proposed. The second component of FOS framework was only proposed conceptually.

(5) Reasonable agreement between P_{max} estimated using the FOS framework and the measurements of hydrofracture pressure could be found through validation.

In future, more data with measurements of hydrofracture pressure and project information may be used for the further validation of SPT-based method and the completion of the FOS framework. Especially for development of the second component of the FOS framework, detailed project information in terms of cost is mandatory.

It would be beneficial to make a comparison between the undrained soil model of the SPT-based method and the Queen's method (Xia, 2009) developed for soft clayey soils. Comparisons between analytical modelling (with application of FOS framework) and numerical modelling may be useful for the improvement of estimation of P_{max}.

References

Aas, G., Lacasse, S., Lunne, T., and Hoeg, K., (1986), "Use of In Situ Tests for Foundation Design on Clay," *Use of In Situ Tests in Geotechnical Engineering*, S.P., Clemence, Ed., GSP 6, American Society of Civil Engineers, New York, pp. 1-30.

Abu-Hejleh, N., O'Neill, M.W., Hanneman, D., and Atwoll, W.J., (2003), "*Improvement of the Geotechnical Axial Design Methodology for Colorado's Drilled Shafts Socketed in Weak Rock,*" Report No. CDOT-DTD-R-2003-6, Colorado Department of Transportation, Denver, 192 p.

Ajayi, L.A. and Balogun, L.A. (1988), "Penetration Testing in Tropical Lateritic and Residual Soils – Nigerian Experience," *Proceedings of the First International Symposium on Penetration Testing*, ISOPT-1, Orlando, March 20-24, 1998, Vol. 1, pp. 315-328.

Anbazhagan, P., and Sitharam, T.G., (2010), "Relationship between Low Strain Shear Modulus and Standard Penetration Test 'N' values." *ASTM Geotechnical Testing Journal*, Vol. 33, No. 2, pp. 150-164.

Anwar, M.B., (2018), "Correlation between PMT and SPT Results for Calcareous Soil," *Housing and Building National Research Center Journal*, Vol. 14, No. 1, pp. 50-55.

API, (2009), – *Recommended Practice on the Rheology and Hydraulics of Oil-Well Drilling Fluids*, API RP 13D, Washington, D.C.

Asperger, M., and Bennett, D., (2011), "The Search for G: Selecting Shear Modulus for Use in Hydrofracture Analysis," *Proceedings of 2011 No - Dig Conference*, Washington, D.C., March 27-31, 2011

Azzouz, A., Baligh, M., and Ladd, C.C., (1983), "Corrected Field Vane Strength for Embankment Design," *Journal of Ge6technical Engineering*, Vol. 109, No. 5, pp. 730-734.

Baroid, (1998), – *Baroid Fluids Handbook*, Chapter 9: Rheology and Hydraulics, Baroid Drilling Fluids, Inc., Haliburton, Houston, Texas.

Barthélemy, H.C., (1974), "An Investigation of Relationships between the SPT and the Cone Penetrometer Resistance in Sands," Project Report, Department of Civil Engineering, Duke University.

Bjerrum, L., (1973), "Problems of Soil Mechanics and Construction on Soft Clays and Structurally Unstable Soils (Collapsible, Expansive and Others)," *Proceedings of the 8th International Conference on Soil Mechanics and Foundation Engineering*, Vol. 3, Moscow, pp. 111-159.

Bowles, J.E., (1996), – *Foundation Analysis and Design*, The McGraw Hill Companies, Inc., Fifth Edition, New York

Bozbey, I., and Togrol, E., (2010), "Correlation of Standard Penetration Test and Pressuremeter Data: A Case Study from Istanbul, Turkey." *Bulletin of Engineering Geology and the Enbrivironment*, Vol. 69, pp. 505–515.

Briaud, J.L., Noubani, A., Kilgore, A., and Tucker, L.M., (1985), "Correlation between Pressuremeter Data and Other Parameters," Research Report, Civil Engineering, Texas A&M University, (Cited from Briaud, J.L., The Pressuremeter, Balkema, A.A., Rotterdam, 1992).

Brown, D.A., Turner, J.P., and Castelli, R.J., (2010), – *Drilled Shafts: Construction Procedures and LRFD Design Methods, NHI Course No. 132014 Geotechnical Engineering Circular NO. 10*, Federal Highway Administration, Report No. FHWA NHI-10-016, Washington, DC.

Callanan, J.F., and Kulhawy, F.H., (1985), ―Evaluation of Procedures for Predicting Foundation Uplift Movements‖, Report EL-4107, Electric Power Research Institute, Palo Alto, Aug. 1985, 124p.

Canadian Geotechnical Society, (2007), – *Canadian Foundation Engineering Manual*, Canadian Geotechnical Society, Fourth Edition, Richmond, British Columbia, Canada

Chandler, R.J., (1988), "The In-Situ Measurement of the Undrained Shear Strength of Clays using the Field Vane," *Vane Shear Strength Testing in Soils: Field and Lab Studies*, STP 1014, ASTM, West Conshohocken, PA, pp. 13-44.

Chen, J.-R., (2004), "Axial Behavior of Drilled Shafts in Gravelly Soils," Ph.D. Dissertation, Cornell University, Ithaca, N.Y.

Cheshomi, A., and Ghodrati, M., (2015), "Estimating Menard Pressuremeter Modulus and Limit Pressure from SPT in Silty Sand and Silty Clay Soils. A Case Study in Mashhad, Iran," *Geomechanics and Geoengineering*, Vol. 10, No. 3, pp. 194-202.

Clayton, C.R.I., (1990), "SPT Energy Transmission: Theory, Measurement and Significance," *Ground Engineering*, Vol. 23, No. 10, pp. 35-43.

Das, B.M., and Sobhan, K., (2013), – *Principles of Geotechnical Engineering*, 8th Ed., Cengage Learning, Stamford, 2013, 362p.

Décourt, L., (1989), "The Standard Penetration Test State-of-the-Art Report," *International Conference on Soil Mechanics and Foundation Engineering*, Rio de Janeiro.

Deltares, (2016), – *D-GEO PIPELINE: Design of Pipeline Installation User Manual*, Version 16.1, Deltares, Delft, 2016, 324p

De Mello, V.F.B., (1971), "The Standard Penetration Test," *4th Panamerican Conference on Soil Mechanics and Foundation Engineering*, San Juan, Puerto Rico, June 1971, Vol. 1, pp. 1-86.

Donovan, N.C., (1968), Personal Communication.

Donovan, N.C., (1969), – Research Brief - Soil Dynamics Specialty Session, *7th International Conference on Soil Mechanics and Foundation Engineering*, Mexico City.

Elwood, D.E.Y., (2008), “Hydraulic Fracture Experiments in a Frictional Material and Approximation for Maximum Allowable Mud Pressure,” M.Sc. Thesis, Queens University, Kingston, Ontario, Canada, July 2008

Goerz, S., Boelhouwer, N., and Taylor, J., (2019), “A New Perspective in Hydrofracture Analysis,” *Proceedings of 2019 No-Dig Conference*, Chicago, Illinois, March 17-20, 2019

Gibbs, H.J., and Holtz, W.G., (1956), “Research on Determining the Density of Sands by Spoon Penetration Testing,” *Proceedings of the 4th International Conference on Soil Mechanics*, London, UK, 1957, Vol. 1, pp. 35-39.

Gross, B.A., (2019), “What’s New at the Geo-Institute?” *Geostrata*, July/August, 2019, pp. 8-9.

Guijt, W., Vrouwenvelder, T., Grensnigt, N., and Dijkstra, G., (2004), “Safety Concept in The New Dutch Pipeline Standard NEN 3650”, *Proceedings of the Fourth International Offshore and Polar Engineering Conference*, Toulon, France, May 23-28, 2004

Hara, A., Ohta, T., Niwa, M., Tanaka, S., and Banno, T., (1974), “Shear Modulus and Shear Strength of Cohesive Soils,” *Soils and Foundation*, Vol. 14, No. 3, Sept. 1974, pp. 1-12.

Hardin, B.O., and Drnevich, V.P., (1970), "Shear Modulus and Damping in Soils: I, Measurement and Parameter Effects, II. Design Equations and Curves," Technical Reports UKY 27-70-CE 2 and 3, College of Engineering, University of Kentucky, Lexington, Kentucky, July.

Hatanaka, M., and Uchida, A., (1996), “Empirical Correlation Between Penetration Resistance and Internal Friction Angle of Sandy Soils,” *Soils and Foundations*, Vol. 36, No. 4, pp. 1-9.

Hettiarachchi, H. and Brown, T., (2009), “Use of SPT Blow Counts to Estimate Shear Strength Properties of Soils: Energy Balance Approach,” *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 135, No. 6, pp. 830-834.

Iai, S., and Kurata, E., (1991), “Pore Water Pressures and Ground Motions Measured during the 1987 Chiba-Toho-Oki Earthquake,” Note of the Prot and Harbour Research Institute, No. 718, pp. 1-18.

Imai, T., and Yoshimura, Y., (1970), “Elastic Wave Velocity and Soil Properties in Soft Soil.” *Tsuchito-Kiso*, Vol. 18, No. 1, pp. 17-22.

Imai, T., and Tonouchi, K., (1982), “Correlation of N-value with S-wave Velocity and Shear Modulus.” *In Proceedings of the 2nd European Symposium on Penetration Testing*, pp. 57-72.

Kenmogne, E., Martin, J.R., and Geofor, S.A., (2011), “Correlation Studies between SPT and Pressuremeter Tests,” *Proceedings of the 15th African Regional Conference on Soil Mechanics and Geotechnical Engineering*, Mozambique, July 18-21, 2011

Kerisel, J., (1964), “Deep Foundations Basic Experimental Facts,” *Cimientos Profundos*, Mexico, Vol. 1, p. 5.

Keulen, B., (2001), “Maximum Allowable Pressures During Horizontal Directional Drillings Focused on Sand,” Master thesis, Delft University of Technology, Delft, Netherlands, September 2001

Kovacs, W.D., Salomone, L.A., and Yokel, F.Y., (1981), “Energy Measurements in the Standard Penetration Test,” *Building Science Series 135*, National Bureau of Standards, Washington, D.C.

Kramer, S.L., (1996), – *Geotechnical earthquake engineering*, Prentice-Hall Inc., New Jersey

Kulhawy, F.H., (1969), “Finite Element Analysis of the Behavior of Embankments,” Ph.D. Dissertation, University of California, Berkeley, 1969, pp. 40-80.

Kulhawy, F.H., Duncan, J.M., and Seed, H.B., (1969), “Finite Element Analysis of Stresses and Movements in Embankments during Construction,” Contract Report S-69-8, U.S. Army Engineers Waterways Experiment Station, Vicksburg, Nov. 1969, 169 p.

Kulhawy, F.H., and Mayne, P.W., (1990), – *Manual on Estimating Soil Properties for Foundation Design*, Electric Power Research Institute EL-6800, Project 1493-6, Electric Power Research Institute, Palo Alto, California.

Kulhawy, F.H. and Chen, J.-R. (2007), “Discussion of ‘Drilled Shaft Side Resistance in Gravelly Soils’ by Kyle M. Rollins, Robert J. Clayton, Rodney C. Mikesell, and Bradford C. Blaise,” *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 133, No. 10, pp. 1325-1328.

Lake, L.M., (1974), “Discussion on Granular Materials,” *Proceedings of Conference on Settlement of Structures*, Cambridge, 1974, Pentech Press, pp. 663.

Lan, H., and Moore, I.D., (2018), “Practical Criteria for Borehole Instability in Sand during Horizontal Directional Drilling,” *Proceedings of 2018 No – Dig Conference*, Palm Springs, California, March 20-24, 2018

Larsson, R., (1980), “Undrained Shear Strength in Stability Calculation of Embankments and Foundations on Clays,” *Canadian Geotechnical Journal*, Vol. 17, No. 4, pp. 591-602.

Liao, S.S.C., and Whitman, R.V., (1986), “Overburden Correction Factors for SPT in Sand,” *Journal of Geotechnical Engineering*, ASCE, Vol. 112, No. 3, pp. 373-337.

Luger, H.J., and Hergarden, H.J.A.M., (1988), “Directional Drilling in Soft Soil: Influence of Mud Pressure,” International Society of Trenchless Technology, *No-Dig Conference*, Washington, D.C., October 16-19, 1988

Marchetti, S., (1980), “In Situ Tests by Flat Dilatometer,” *Journal of Geotechnical Engineering Division*, ASCE, Vol. 106, No. GT3, Mar. 1980, pp. 299-321.

Matsushita, K., Kishida, H., and Kyo, K., (1967), "Experiments on Damping of Sands," *Transactions of the Architectural Institute of Japan*, Summaries of Technical Papers (Annual Meeting of AIJ, 1967), p. 166.

Mayne, P.W., and Frost, D.D., (1989), “Dilatometer Experience in Washington, D.C., and Vicinity,” *Transportation Research Record 1169*, Transportation Research Board, Washington, D.C., 1989, pp. 16-23.

Mayne, P.W., and Kemper, J.B., (1988), “Profiling OCR in Stiff Clays by CPT and SPT,” *ASTM Geotechnical Testing Journal*, Vol. 11, No. 2, pp. 139-147.

Mayne, P.W., and Mitchell, J.K., (1988), “Profiling of Overconsolidation Ratio in Clays by Field Vane,” *Canadian Geotechnical Journal*, Vol. 25, No. 1, pp. 150-157.

Mayne, P.W., and Frost, D.D., (1989), “Dilatometer Experience in Washington, D.C., and Vicinity,” *Transportation Research Record 1169*, Transportation Research Board, Washington, D.C., 1989, pp. 16-23.

Mesri, G., (1989), “A Re-evaluation of $S_{u(mob)} = 0.22\sigma'_p$ Using Laboratory Shear Tests,” *Canadian Geotechnical Journal*, Vol. 26, No. 1, pp. 162-164.

Meyerhof, G.G., (1956), “Penetration Tests and Bearing Capacity of Cohesionless Soils,” *Journal of Soil Mechanics and Foundation Division*, ASCE, Vol. 82, No. SM1, Jan. 1956, pp. 1-19.

Meyerhof, G.G., (1976), "Bearing Capacity and Settlement of Pile Foundations," *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 102, No. GT3, March, pp. 197-228.

Miller, M.A., and Robinson, J.L., (2019), “Soil Mechanics and Calculating Hydraulic Fracture Risk,” *Proceedings of 2019 No-Dig Conference*, Chicago, Illinois, March 18-21, 2019

Morris, P.H., and Williams, D.J., (1993), "A New Model of Vane Shear Strength Testing in Soils," *Geotechnique*, Vol. 43, No. 3, pp. 489-500.

Morris, P.H., and Williams, D.J., (1994), "Effective Stress Vane Strength Correction Factor Correlations," *Canadian Geotechnical Journal*, Vol. 31, No. 3, pp. 335-342.

Murray, C.D., Osbak, M., and Bayat, A., (2013), "Horizontal Directional Drilling – Construction Risk Management Strategies," *Proceedings of 2013 Pipelines Conference*, ASCE, Fort Worth, Texas, June 23-26, 2013

Nassaji, F., and Kalantari, B., (2011), "SPT capability to estimate undrained shear strength of fine-grained soils of Tehran, Iran," *Electronic Journal of Geotechnical Engineering*, Vol. 16, 2011, pp. 1229-1238.

NEN, (2006), NEN 3650-1:2003+A1:2006. Eisen voor buisleidingsystemen - Deel 1: Algemene – Katern 1 tot en met 6 (Requirements for pipeline systems - Part 1: General – Quire 1 to 6), in English

NEN, (2017), NEN 3650-1+C1:2017. Eisen voor buisleidingsystemen - Deel 1: Algemene eisen (Requirements for Pipeline Systems - Part 1: General Requirements), in English

Newcomb, D.E., and Birgisson, B., (1999), – NCHRP Synthesis of Highway Practice 278: Measuring In Situ Mechanical Properties of Pavement Subgrade Soils, Transportation Research Board, National Research Council, Washington D.C., 1999.

Nixon, I.K., (1982), "Standard penetration test: state of the art report," *Proceedings of the Second European Symposium on Penetration Testing*, Vol. 1, Amsterdam, 1982, pp. 3-24.

Ohba, S., and Toriumi, I., (1970), "Research on Vibration Characteristics of Soil Deposits in Osaka, Part 2, on Velocities of Wave Propagation and Predominant Periods of Soil Deposits." Abstract, Technical Meeting of Architectural Institute of Japan.

Ohsaki, Y., and Iwasaki, R., (1973), "On Dynamic Shear Moduli and Poisson's Ratio of Soil Deposits." *Soils and Foundations*, Vol. 13, No. 4, pp. 61-73.

Ohta, T., Hara, A., Niwa, M., and Sakano, T., (1972), "Elastic Moduli of Soil Deposits Estimated by N-values." *In Proceedings of the 7th Annual Conference, The Japanese Society of Soil Mechanics and Foundation Engineering*.

Ohya, S., Imai, T., and Matsubara, M., (1982), "Relationships between N-value by SPT and LLT measurement results," *Proceedings of the Second European Symposium on Penetration Testing*, Vol. 1, Amsterdam, 1982, pp. 125–130.

Olsen, R.S., (1997), "Cyclic Liquefaction based on the Cone Penetrometer Test," *Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils*, National Center for Earthquake Engineering Research, Dec. 1997, State University of New York at Buffalo, pp. 225-276.

O'Neill, M.W., and Reese, L.C., (1999), – *Drilled Shafts: Construction Procedures and Design Methods*. Publication No. FHWA-IF-99-025, Federal Highway Administration, Washington, D.C., p. 758.

Park, I., and Bayat, A., (2020), "Simplified Application of the Delft Method to Estimate Maximum Allowable Annular Pressure in HDD", *Proceedings of 2020 Pipelines Conference*, ASCE, San Antonio, Texas (Conference Held Virtually), August 9-12, 2020

Peck, R.B., Hanson, W.E., and Thorburn, T.H., (1974), – *Foundation Engineering*, 2nd Ed., John Wiley and Sons, New York, 1974, 310p

Poulos, H.G., (1978), "Normalized Deformation Parameters for Kaolin," Research Report R336, School of Civil Engineering, University of Sydney, pp. 18.

Riggs, C.O., (1986), "North American Standard Penetration Test Practice - An Essay," *Proceedings of ASCE Conference on Use of In-situ Tests in Geotechnical Engineering*, Blacksburg, p. 945-965.

Rostami, A., (2017), "Prediction and Evaluation of Annular Pressure in Horizontal Directional Drilling," Ph.D. Thesis, University of Alberta, Edmonton, Alberta, Canada, April 2017

Sabatini, P.J., Bachus, R.C., Mayne, P.W., Schneider, J.A., and Zettler, T.E., (2002), "Evaluation of Soil and Rock Properties," *Geotechnical Engineering Circular No. 5*, Federal Highway Administration, Washington, D.C., p. 385.

Sanglerat, G., (1972), – *The Penetration and Soil Exploration; Interpretation of Penetration Diagrams – Theory and Practice*, Elsevier Publishing Co, Amsterdam

Sanglerat, G., and Sanglerat, T.R.A., (1982), "Pitfalls of the SPT," *Proceedings of 2nd Europe Symposium on Penetration Testing*, Amsterdam, May 24-27, 1982, Vol. 1, pp. 143-145.

Schmertmann, J.H., (1975), "Measurement of In-Situ Shear Strength," *Proceedings of ASCE Specialty Conference on In-Situ Measurement of Soil Properties*, Vol. 2, Raleigh, 1975, pp. 57-138. (Closure: pp. 175-179)

Schmertmann, J. H., and Palacios, A., (1979), "Energy Dynamics of SPT," *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 105, pp. 909-926.

Seed, H. B., (1968), Unpublished Test Results.

Seed, H.B., and Idriss, I.M., (1970), "Soil Moduli and Damping Factors for Dynamic Response Analysis," Report EERC70-10.

Seed, H.B., Idriss, I.M., and Arango, I., (1983), "Evaluation of Liquefaction Potential Using Field Performance Data," *Journal of the Geotechnical Engineering Division*, Vol. 109, No. 3, pp. 458-482.

Seed, H.B., Tokimatsu, K., Harder, L.F. and Chung, R.M., (1985), "Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations," *ASCE Journal of Geotechnical Engineering*, Vol. 111, No. 12, pp. 1425-1445.

Silver, M.L., and Seed, H.B., (1969), "The Behavior of Sands under Seismic Loading Conditions," Report No. EERC 69-16, University of California, Berkeley.

Sivrikaya, O., and Toğrol, E., (2002), "Relations between SPT-N and q_u," *5th Intern. Congress on Advances in Civil Engineering*, Istanbul, Turkey, 2002, pp. 943–952.

Sivrikaya, O., and Toğrol, E., (2006), "Determination of undrained strength of fine-grained soils by means of SPT and its application in Turkey", *Engineering Geology*, Vol. 86, No. 1, 2006, pp. 52-69.

Sivrikaya, O., (2009), "Comparison of Artificial Neural Networks Models with Correlative Works on Undrained Shear Strength", *Eurasian Soil Science*, Vol. 42, No. 13, 2009, pp. 1487-1496.

Skempton, A.W., (1948), "Vane Tests in the Alluvial Plain of River Forth near Grangemouth," *Geotechnique*, Vol. 1, No. 2, pp. 111-124.

Skempton, A.W., (1986), "Standard Penetration Test, Procedures and Effects in Sands of Overburden, Relative Density, Particle Size, Aging and Over-consolidation," *Geotechnique*, Vol. 36, No.3, pp. 425-447.

Sowers, G.F., (1979), – *Introductory Soil Mechanics and Foundations: Geotechnical Engineering*, Macmillan, Fourth Edition, New York

Staheli, K., Bennett, D., O'Donnell, H.W., and Hurley, T.J., (1998), "Installation of Pipelines beneath Levees Using Horizontal Directional Drilling," CPAR-GL-98-1, April 1998

Staheli, K., Price, C.G., and Wetter, L., (2010), "Effectiveness of Hydrofracture Prediction for HDD Design," *Proceedings of 2010 No - Dig Conference*, Chicago, Illinois, May 2-7, 2010

Stroud, M.A., (1974), "The Standard Penetration Test in Insensitive Clays and Soft Rocks," *Proceedings of the European Symposium on Penetration Testing*, Stockholm, June 5-7, 1974, Vol. 2, No. 2, pp. 367-375.

Tavenas, F.A., Capelle, J.F., and La Rochelle, P., (1970), "Étude des Sables submergés par échantillonnage non remanié," (Study of Submerged Sands by Undisturbed Sampling), *Canadian Geotechnical Journal*, Vol. 7, No. 1, p. 37.

Tavenas, F.A., (1971), "Discussion to de Mello (1971)," *4th Panamerican Conference on Soil Mechanics and Foundation Engineering*, Puerto Rico, Vol. III, pp. 64-69.

Terzaghi, K. and Peck, R.B., (1967), – *Soil Mechanics in Engineering Practice*, John Wiley and Sons, Second Edition, New York

Terzaghi, K., Peck, R.B., and Mesri, G., (1996), – *Soil Mechanics in Engineering Practice*, John Wiley and Sons, Third Edition, New York

Trautmann, C.H., and Kulhawy, F.H., (1987), "CUFAD – A Computer Program for Compression and Uplift Foundation Analysis and Design," Report EL-4540-CCM, Vol. 16, Electric Power Research Institute, Palo Alto, Oct. 1987, 148p.

Uchida, A., Kano, H., and Tokimatsu, K., (1990), "Correlation between Dynamic Properties and Static Properties of Sands obtained by In-Situ Freezing," *25th Annual Meeting of JSSMFE*, Vol. 1, pp. 767-768.

USACE, (2007), Hurricane and Storm Damage Reduction System Design Guidelines. Interim., U.S. Army Corps of Engineers, New Orleans District, October 23, 2007

Van Brussel, G.G., and Hergarden, H.J.A.M., (1997), “Research CPAR program: Installation of Pipelines beneath Levees Using Horizontal Directional Drilling”, Dept. of Foundations and Underground Engineering, Delft Geotechnics, Delft, Netherlands, pp. 1-16.

Verruijt, A., (1993), – *Grondmechanica*, 4th Ed., Delftse Uitgevers Maatschappij, Delft, Netherlands

Vesic, A.S., (1967), “A Study of Bearing Capacity of Deep Foundations,” Final Report, Georgia Institute of Technology, Atlanta, Georgia

Weissman, G.F., and Hart, R.R., (1961), "The Damping Capacity of Some Granular Soils," *Symposium on Soil Dynamics*, ASTM Special Technical Publication No. 305, June 1961, pp. 45-54.

Wroth, C.P., (1975), “In-Situ Measurement of Initial Stresses and Deformation Characteristics,” *Proceedings of ASCE Specialty Conference on In-Situ Measurement of Soil Properties*, Vol. 2, Raleigh, pp. 180-230.

Xia, H., (2009), “Investigation of Maximum Mud Pressure within Sand and Clay during Horizontal Directional Drilling,” Ph.D. Thesis, Queens University, Kingston, Ontario, Canada, January 2009

Yagiz, S., Akyol, E., and Sen, G., (2008), “Relationship between the Standard Penetration Test and the Pressuremeter Test on Sandy Silty Clays: A Case Study from Denizli.” *Bulletin of Engineering Geology and the Environment*, Vol. 67, pp. 405-410.

Yoshimi, Y., (1994), “Relationship among Liquefaction Resistance, SPT N-value and Relative Density for Undisturbed Samples of Sands,” Tsuchi-to-Kiso, JSSMFE, Ser. No. 435, Vol. 42, No. 4, pp. 63-37.

Yu, H.S., and Houlsby, G.T., (1991), “Finite Cavity Expansion in Dilatant Soils: Loading Analysis,” *Geotechnique*, 41, No. 2, pp. 173-183.