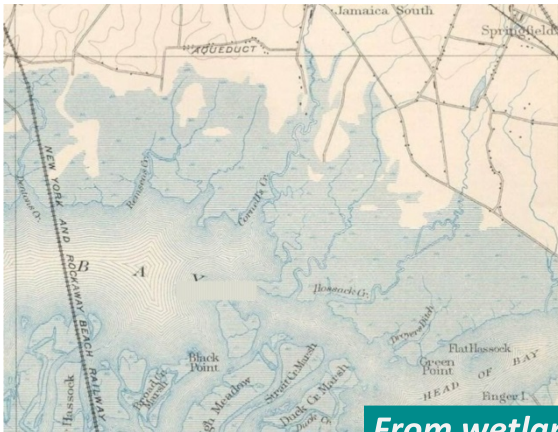
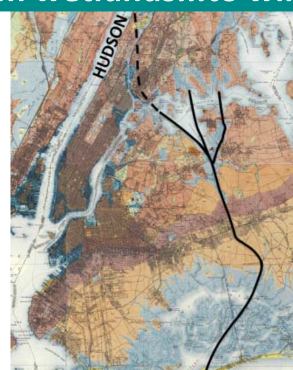


From wetlands...to wings...



and a river ran through it...

John F. Kennedy International Airport: A Seven-Decade Case Study of the Evolution of Geotechnical and Foundation Engineering Design and Construction Practice

A White Paper
by
John S. Horvath, Ph.D., P.E., LifeM.ASCE
d/b/a
John S. Horvath Consulting Engineer
Scarsdale, New York, U.S.A.

November 2014

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John S. Horvath, Ph.D., P.E., LifeM.ASCE
d/b/a
John S. Horvath Consulting Engineer
Scarsdale, New York, U.S.A.

Website: jshce.com
Email: jsh@jshce.com

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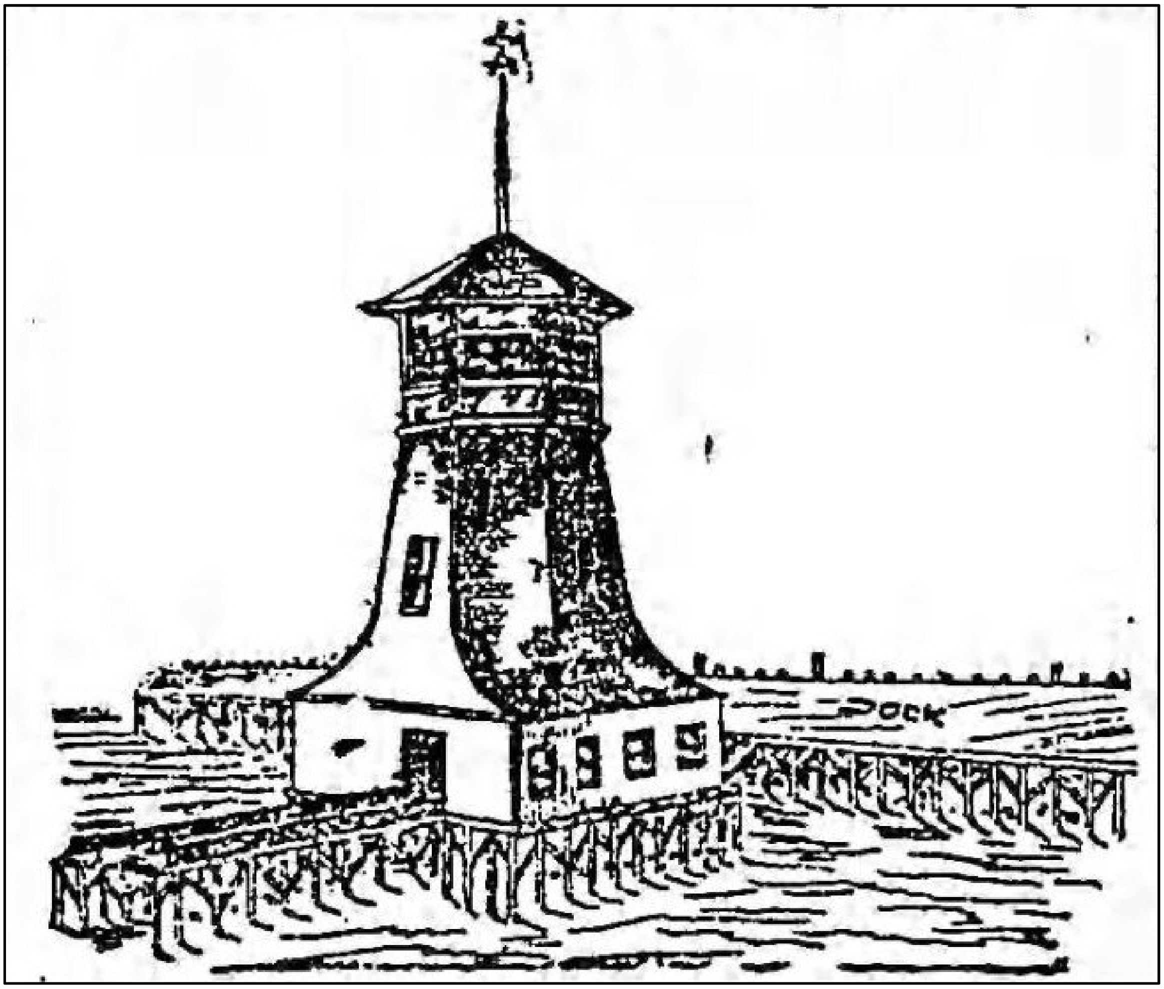

Leon(h)ard Eppig's Lighthouse/Observatory/Water Tower (Tank) at Idlewild Park
Idlewild (Long Neck) Point - Head of Broad Channel in Jamaica Bay
Queens, New York
[*The Brooklyn Daily Eagle* - 11 July 1897]

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Preface

This white paper is broadly related to my long-standing research interest in integrating the process of soil-property determination from site characterization with estimates of shallow and deep foundation performance (settlement and bearing capacity) into one, seamless analytical algorithm. This interest has its roots in several foundation-related prediction symposia held in the late 1980s and early 1990s in the U.S. to which I contributed while I was a member of the faculty of Manhattan College in New York City.

However, the specific subject of this paper actually has its genesis well before that, literally in the first few weeks of my full-time professional employment that began in June 1972 with the Port Authority of New York and New Jersey (PANYNJ). As a young, entry-level engineer fresh out of school with a Master degree in civil/geotechnical engineering I was assigned to work on a project to evaluate different types of driven piles for potential use to support a proposed parking garage within the Central Terminal Area (CTA) for commercial/civilian passenger traffic at the John F. Kennedy International Airport (JFKIA) in New York City. Although JFKIA had been in existence for approximately 30 years at that point in time and there was substantial experience with driven piles (the foundation of choice for virtually all buildings and transportation-related structures there then and now), the then-Soils and Foundations Division of the PANYNJ Engineering Department had already developed a well-deserved reputation for always looking to push the edge of the technological envelope whenever a new major project came about. This reputation had been established and burnished in the 1960s by the late Martin S. 'Marty' Kapp who, among other things, brought many geotechnical advances to U.S. geotechnical and foundation engineering practice as part of the original World Trade Center project. Marty Kapp's legacy was maintained and furthered by Donald L. 'Don' York, P.E. who was the head of the Soils and Foundations Division when I joined the PANYNJ. In later years (1987-2014), during my tenure on the faculty of Manhattan College, Don York graciously provided me with additional data concerning driven piles at JFKIA for my academic research and instruction purposes.

This early-career exposure to driven piles in general and tapered piles in particular left a lasting impression on me throughout my professional career. In 2012, it occurred to me that the geotechnical engineering experiences at JFKIA, which by then had reached the 70-year mark, provided a rather unique opportunity to trace the evolution of modern geotechnical and foundation engineering. Simply stated, JFKIA represented an engineered facility with a single owner where the basic needs (to provide global commercial air-transport capability) had not changed over time. However, how these needs were viewed and satisfied from the perspective of the civil engineer specializing in geotechnical and foundation engineering had changed enormously, not just with respect to deep foundations but also in areas such as site characterization and earthquake engineering. This, then, is the intended goal of this paper, to define and describe the geotechnical and foundation engineering needs at JFKIA and to discuss and synthesize how design and construction technology on all levels has developed and evolved over the seven-plus decades since construction of JFKIA first began.

John S. Horvath, Ph.D., P.E., LifeM.ASCE
Scarsdale, New York, U.S.A.
18 November 2014

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Executive Summary

When viewed from the perspective of technical needs, a geotechnical or foundation engineering case history for a constructed facility has three interactive aspects:

- Definition of these technical needs that is a unique, project-specific combination of owner requirements for the proposed constructed facility and site geology created by nature as well as any prior human modification.

- Assessment of these defined needs by licensed design professionals who ultimately craft the geotechnical or foundation aspects of a design alternative for the proposed constructed facility. In some organizations and situations there may be formal peer review and/or value engineering of the proposed design alternative.

- Fulfillment (execution) of the geotechnical or foundation design by a construction contractor who may bring their own expertise and experience to bear in terms of approved design modifications and/or alternatives.

Construction of what is now known as the John F. Kennedy International Airport (JFKIA) in New York City began in April 1942. From the beginning it was planned to be the premier commercial international airport for both passenger and freight traffic within the geographical boundaries of the City. In that sense the airport has been a complete and unqualified success as it has held that position from the time it officially began commercial flight operations in July 1948 to the present.

When viewed within the framework of the above-described tripartite model for technical needs, JFKIA presents a unique case history in geotechnical and foundation engineering as for more than 70 years now it has essentially had the same owner and the same basic technical need of providing all the necessary ground facilities and services related to commercial passenger and freight aviation on an international scale. With this variable eliminated, it is possible to focus on how the design and construction sides of engineered construction have combined to fulfill these more or less unchanging defined needs over time.

The outcome of this exercise, which is presented in detail in this paper, actually provides a broad, insightful view of how geotechnical and foundation engineering design and construction practice has changed in the U.S. from a time on the cusp of World War Two when modern soil mechanics was still in its infancy all the way to the relatively sophisticated present. This change turns out to have taken two broadly different forms:

- *Development of different ways to do that which has always been done*. With specific reference to JFKIA, humans have been driving piles for foundation support for thousands of years. Until the last 100 years or so these piles were always timber piles which are naturally tapered. Deep foundations have always been used at JFKIA and driven tapered piles have, with few exceptions, always been the piling alternative of choice. However, axial-compressive design resistances (allowable capacities) per pile have increased by more than a factor of 10 over the years due to a wide variety of technological developments and advances. In addition, other deep-foundation alternatives based on drilling as opposed to driving have been developed over the years and found to be usable at JFKIA although they have not been exploited to date.

- *Appearance and evolution of entirely new design issues*. Like a new island arising out of the ocean due to the buildup of magma emanating from within the Earth's interior, over time entirely new technical considerations arise and evolve that must be considered routinely on projects by design professionals. In most cases these new developments represent phenomena or issues that have always been there (as that magma has always been within the Earth) but for various reasons never rose to the level of prominence that exists at present. For example, it is hard to imagine but construction of JFKIA began well before the geological concept of plate tectonics and the concomitant modern understanding of seismicity was accepted no less widely known by geoprofessionals. Thus while earthquakes have been known to humans since antiquity the fact that their direct (seismic shaking) and indirect (seismic liquefaction) consequences need to be considered in the New York City metropolitan area in general, and at JFKIA in particular, is something that design professionals have only recognized in recent years, well after the basic form and function of JFKIA had been completed. Similarly, human existence has always impacted nature in various negative ways. However, the formal consideration of environmental issues and concomitant adverse human impacts is also something that has become a requirement only in recent years. With specific regard to JFKIA, it is quite possible the airport would never have been constructed in the first place in the current regulatory and environmentally conscious environment. This is because virtually all of the approximately 5,000 acres (2,000 hectares) that comprise the current JFKIA were created by hydraulic filling over what was largely virgin tidal wetlands and an adjacent brackish tidal bay that today are collectively recognized as being so environmentally significant that they are part of a national park and wildlife refuge.

In summary, while this paper is limited to developments at JFKIA most of the observations made are applicable to the global practice of geotechnical and foundation engineering design and construction. Thus in addition to the very detailed presentation of material related specifically to JFKIA this paper includes a discussion of how lessons learned at JFKIA can be applied or used on a global scale for a wide variety of constructed facilities.

BACKGROUND

Introduction

This paper is unusual as scholarly publications go because it has, to a certain extent, been more than 40 years in the making. This is because the content, both factual and professional opinion, presented herein has been under development episodically since June 1972 when the writer began a professional-engineering career as an entry-level civil engineer in the former Soils Division of The Port Authority of New York and New Jersey (PANYNJ)[1] Engineering Department.

The writer's first assignment back then was to participate in both the field inspection and office-analytical components of a test-pile[2] program involving various types of driven piles that was just commencing at the John F. Kennedy International Airport (JFKIA). This airport is located in the extreme southeastern corner of Queens County in the State of New York which is also the Borough of Queens in the City of New York. JFKIA has been the primary international commercial airport for both passenger and freight traffic serving the New York City (NYC) metropolitan area since commercial flight operations began there in July 1948. The location of JFKIA relative to nearby geographic locations is shown in Figure 1.

This 1972 test-pile program was the writer's introduction to what has turned out to be a career-long professional involvement with projects and research involving JFKIA as well as driven piles in general and tapered piles (from the very beginning the deep foundation of choice at JFKIA) in particular that has now spanned more than five decades. In particular, this initial project involvement at JFKIA provided the writer with both an appreciation of and intellectual curiosity about the unique behavior and analytical uncertainty of tapered piles under friction-pile (floating-pile) conditions in coarse-grain soil. This eventually led to a number of prior publications (Horvath 2002, 2003a, 2003b, 2003c, 2004; Horvath and Trochalides 2004; Horvath et al. 2004a, 2004b)[3] and still remains an area of professional interest of the writer.

Purpose and Scope of Paper

The particular motivation (actually more of an inspiration) to write this paper at this point in time was the announcement in 2012 that the Driven Pile Committee of the Deep Foundations Institute (DFI)[4] was in the process of creating a book containing case histories dealing with driven piles. This caused the writer to reflect at length about deep-foundation experiences at JFKIA.

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[1] This bi-state agency was originally called the Port of New York Authority (PONYA) when created by interstate compact in 1921. The name was changed in 1972 to better reflect the dual-state nature of the Authority's many and varied commercial, marine, and air and ground transportation facilities.

[2] Throughout this paper, the term 'pile' if used alone is in the narrow context meaning 'driven pile' as is consistent with colloquial U.S. usage. Note that this contrasts with the broader definition and usage of the term 'pile' in many other countries and regions to generically mean any type of deep foundation. However, where it is judged desirable to avoid any ambiguity the term 'driven pile' is used in this paper explicitly instead of simply 'pile'.

[3] Cited references for complete documents, including URLs for digital versions available on the Web, are listed in a separate Reference section at the end of this paper.

[4] The writer is a Charter Independent Individual Member of DFI. The writer's early-career involvement with driven piles at JFKIA in 1972 was the primary motivation for the writer's joining DFI at that organization's inception in 1976.

Figure 1. General Location Map.

The outcome of the writer's reflection was the realization that JFKIA represents a relatively unique case history subject in geotechnical and foundation engineering for several reasons:

- It has been in existence (including initial construction time) for over 70 years. This combination of length of time and timeframe encompasses the bulk of the existence of modern soil mechanics that the writer defines as appearing in Europe in the 1920s and globally by the 1930s when the first international conference was held. More broadly, it spans a period of time that began with the sliderule being the primary computational tool available to the civil engineer. Foundation-wise this mattered little when JFKIA was first being constructed as foundation design of the early 1940s was still more art than science and was based largely on local experience and precedence, with pile driving (dynamic) formulas the only analytical tool for estimating the installed axial-compressive geotechnical resistance of driven piles. Moreover, what we now call geotechnical engineering (the term itself did not exist in the 1940s) was, to a certain extent, viewed as a novelty by some if not many. There was prevailing opinion that existed well into the second half of the 20th century that soil mechanics, as it was then called, did not need to be a separate discipline within civil engineering but simply remain a subset of structural engineering as it had been historically. This is reflected in the fact that what we now call geotechnical engineering was not taught as a required subject at the undergraduate level to the vast majority of students, at least in the U.S., until the latter decades of the 20th century. Keep in mind that Terzaghi's seminal English-language text, *Theoretical Soil Mechanics*, was not published until 1943, a year after initial construction of JFKIA began, and the first edition of Terzaghi and Peck's equally seminal *Soil Mechanics in Engineering Practice* was not published until 1948, the year flight operations officially began at JFKIA.

- It covers a relatively large area, with a surface area currently reported[5] as 4,930 acres[6] (1,995 hectares)) or 7.7 square miles (20 km^2). However, despite the relatively large physical area subsurface conditions are remarkably uniform throughout virtually the entire airport which means that basic foundation needs are essentially the same almost everywhere within the airport property.

- It has had the same use (primarily international, commercial (civilian) passenger and freight aviation) for its entire existence that means the basic types of structures (terminals, hangers, and various types of support buildings plus bridges and elevated roadways, taxiways, and transitways) requiring foundation support have not varied significantly for its entire existence.

- It has had the same owner[7] (PANYNJ) for its entire operational existence. Of significance and relevance is that the PANYNJ is an unusual airport owner in that it once had a very large Engineering Department that either did foundation designs in-house or exhibited strong peer-review control of foundation designs performed by private consulting firms for terminals and other facilities constructed by individual airlines. Furthermore, the PANYNJ Engineering Department had in-house access to a state-of-art mainframe computer system for performing relatively advanced, sophisticated engineering analyses by circa 1970, a time when very few engineering practitioners outside of the aerospace industry had access to such computational tools. Even though the size of the PANYNJ's Engineering Department has been much reduced in recent years it still exhibits significant control over foundation design and construction at the airport. However, the most significant aspect of PANYNJ ownership from the perspective of this paper is that their Engineering Department in general, and former Soils Division in particular, was, especially in the past during the Kapp-York leadership years spanning the 1960s to 1990s, very progressive and innovative. The first-in-the-U.S. use of a slurry (structural-diaphragm) wall for the 'bathtub' wall at the original (1960s) World Trade Center site is perhaps the best and most widely known example of this. Less well known but of significance and relevance to this paper is that the PANYNJ was an early user of the wave-equation software for assessing pile driving during all phases of a project, include design (they were using the original wave-equation program developed by the Texas Transportation Institute (TTI) of Texas A&M University in the late 1960s when the writer joined the PANYNJ in 1972). In addition, they are believed to have been the first user of what was then called the Case-Goble (or simply Case) Method of dynamic

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[5] www.panynj.gov/airports/jfk-facts-info.html. Accessed 31 October 2014.

[6] U.S. practice for the use of punctuation with numbers is followed throughout this paper. Specifically, a decimal point is used to separate a fractional part of a number from its integer part. A comma is used to separate the integer part of a number into three-digit groupings.

[7] The term 'owner' is used here in a broad, not literal, sense. As discussed subsequently, the JFKIA property is actually owned by the City of New York but has been operated by the PANYNJ under lease with and from the City since 1947, five years after construction began but a year before commercial flights officially commenced. Furthermore, the PANYNJ has controlled all development and construction at JFKIA since it began its lease with little, if any, apparent input, oversight, or control by the City. Thus the PANYNJ has functioned in every meaningful sense of the word as the airport owner although it was not involved in the initial planning and construction stages.

measurements of pile driving[8] in the NYC metropolitan area when they retained Goble-Rausche-Likins and Associates[9] to use this then-novel methodology at JFKIA for the aforementioned test-pile program in Summer 1972. The PANYNJ is also believed to have been among the first in the NYC metropolitan area to use the cone penetrometer test (CPT) for site characterization (at JFKIA as it turns out, in the late 1980s) as well as to perform seismic-liquefaction[10] assessments at JFKIA.

In conclusion, in the writer's opinion JFKIA presents a rather unique case study for geotechnical and foundation engineers of how all the many and varied aspects of geotechnical engineering in general, and deep-foundation design and construction practice in particular, have evolved to satisfy a more or less fixed set of technical needs (demands) in a classical friction-pile scenario for the same owner over a period of eight decades that spans nearly the entire history of modern geotechnical and foundation engineering to date. Although the specific combination of details may be unique to JFKIA, the writer feels that there is sufficient generality and universality in the individual details to make them of value to geotechnical and foundation engineers worldwide. It is this diversity of potential use that is the underlying purpose for the writer's preparing this paper as a pro-bono contribution to the good-of-the-order of geotechnical and foundation design and construction.

Terminology

Of all the major disciplines and areas of specialization within the profession of civil engineering, foundation engineering stands out for its lack of standardization in terms of both variable (parameter) notation and terminology. Nowhere is this this lack of standardization more apparent than with deep foundations in general and driven piles in particular. Thus it is important in written work, especially documents such as this that are intended for an international audience, to clearly define both notation and terminology.

The basic rule of thumb adopted by the writer with regard to both notation and terminology in this paper is to use consistent terminology for a parameter, material property, etc. based on what, in the writer's experience and opinion, is most commonly used colloquially in U.S. practice at the present time. One exception is that, if necessary for the intended purposes of this paper, an older, perhaps even now-deprecated, term will be used for its historical significance although there are instances (that are noted) where this is not done to avoid even mention of something that is now considered to be unacceptable.

However, even with this self-imposed guideline there is still substantial variability when it comes to deep foundations as even within current U.S. practice there is a divergence and variation of terminology. For example, and to cite an extreme case in this regard, the lower end of an installed deep-foundation element is variously referred to in the U.S. as the base, bottom, end, point, tip, and toe. Therefore, the writer has chosen to adopt the suggestions/recommendations of Fellenius (1999b) as summarized in Figure 2 as they seem reasonable in that they are at least among the terms the writer has seen and heard used in U.S. practice plus have the added benefit of being directly translatable into other languages. So, for example, toe, which is certainly universal in its meaning in any language, will be used for the lower end of an installed pile.

[8] This system has evolved over the past 40-plus years to the *Pile Driving Analyzer®* (*PDA*) and associated technologies sold by Pile Dynamics, Inc.
[9] Now GRL Engineers, Inc.
[10] Throughout the remainder of this paper, the term 'liquefaction' when used alone will mean 'seismic liquefaction'.

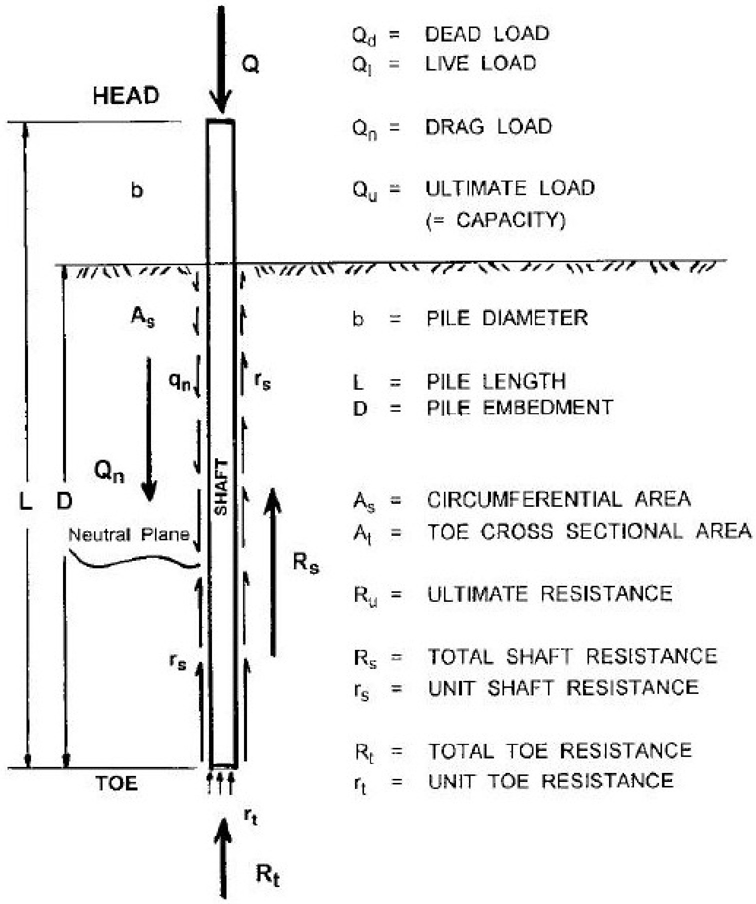

Figure 2. Pile Terminology [from Fellenius 1999b].

However, it should be recognized that no matter how well-intentioned efforts such as those promulgated by Fellenius may be there are simply terms and practices that are firmly entrenched in U.S. foundation engineering and construction practice that the writer believes are not going to change anytime soon. For example, the head of a timber pile is universally called the butt and pile resistances (capacities) are still quoted in the Imperial (U.S. customary) unit of tons (1 ton = 2,000 pounds = 8.9 kN) as opposed to kips or pounds.

OVERVIEW

From the perspective of technology, engineered construction can be viewed as consisting of an interaction between and among three distinct components of technical need (demand) as depicted by the tripartite model shown pictorially in Figure 3:

- **Definition** as dictated both by nature (site conditions, which may or may not have been altered by prior human activity) and humans (the project client/owner, which are not necessarily the same entity, and any other stakeholders such as funding agencies who have design peer-review or other project-oversight capability).

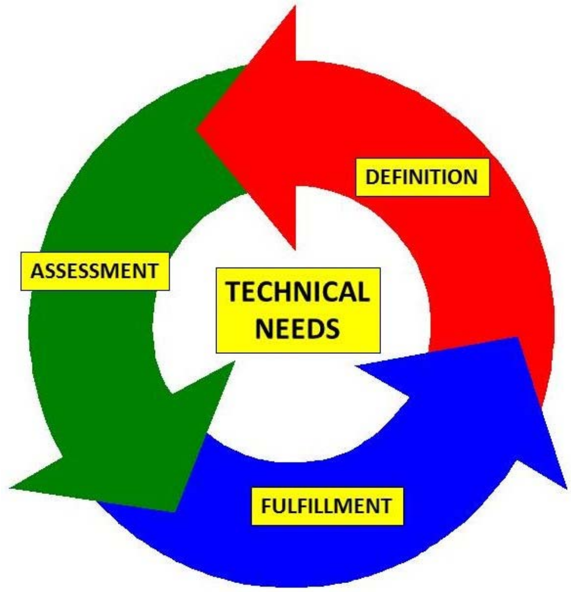

Figure 3. Trilogy of Technical Project Needs in Engineered Construction.

- **Assessment** by the licensed design professionals involved in the project who develop a design concept to satisfy the defined human needs within the context of existing site conditions, all based on the prevailing state-of-knowledge and state-of-practice as deemed achievable within any local constructability constraints.

- **Fulfillment** that consists not only of design execution by construction contractor(s) but also compliance verification (defined here as c*onstruction quality assurance*, CQA) by an organizational entity separate from the contractor(s) to ensure that the design has been executed as intended as defined by the project documents (plans and specifications). CQA should not be confused with *construction quality control* (CQC) which is something the contractor(s) and material supplier(s) involved in a project should implement within their organizations as basically a pre-CQA initiative so that, ideally at least, no exceptions will be taken when CQA is conducted.

It is important to note that each of these three components plays an equally vital role in the overall outcome and performance of a given project. As with a three-legged stool, long-term satisfactory project performance requires that each leg be equal in its length, i.e. contribution to the synergistic outcome of the overall project.

In general, the many and diverse elements that make up each of these three components is temporally dynamic and ever-changing due to the evolution, sometimes relatively rapid, of both technological (knowledge increase) as well societal (acceptable risk, economics, environmental acceptance, human comfort, etc.) factors. In addition, because each of these three components interacts with and affects the other two, the overall net result will also be temporally dynamic and in a way that is unique to each project.

The remainder of this paper is devoted to a detailed discussion of each of these three components of technical need as they relate to the geotechnical and foundation engineering history at JFKIA. This is followed by a presentation of thoughts and opinions in the following three areas based on the writer's 42-plus years of professional engineering practice and observation of the evolution of foundation design and construction practice at JFKIA:

- a summary to highlight key foundation-related issues and developments from the JFKIA experiences that are relevant to geotechnical and foundation engineering practice in general;

- a commentary concerning past and present trends; and

- thoughts about possible future trends.

TECHNICAL NEEDS: DEFINITION

Introduction

This section of the paper deals with how the circa-1940 topography and geology (including human modifications made up to that point in time) in the extreme southeastern corner of NYC impacted and interacted with the airport development plans of mid-20th century humans to define a unique set of site- and project-specific foundation needs for JFKIA.

Overview

The single most important factor defining the foundation-related needs at JFKIA is that most of the airport property consists of made-land (landfill) created by hydraulically placing sand over a brackish-water tidal wetlands (marine tidal marsh, MTM) consisting of peat and/or organic clay as well as within adjacent open-water areas underlain by organic clay, all of which were within the northeast periphery of Jamaica Bay, a brackish-water extension of the Atlantic Ocean (see Figures 1 and 4). This combination of uncontrolled[11] fill overlying organic soils meant, at the time of the original airport construction in the 1940s, that any structure of significance had to be supported on deep foundations that bypassed both the uncontrolled fill and organic soils as these materials would have nominally zero bearing value from code perspectives. The extensive suite of ground modification/improvement geotechnologies[12] developed in recent decades that might have allowed for alternative foundation strategies to be considered did not, for all practical purposes, exist at the time.

Because of the precedence of using deep foundations, and the practical impossibility of performing ground modification for new structures built in recent years that are adjacent to older structures supported on deep foundations and overlying unimproved ground, this has meant that the basic foundation-design strategy of installing deep foundations and bypassing the surficial fill and underlying organic strata has continued to the present.

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[11] In this context, 'uncontrolled' means the lack of what nowadays would be considered normal good practice with regard to controlling both the gradation of soil-particle sizes placed as well as the either the relative density or relative compaction of the placed material. Historically, it has been important within the context of the NYC building code to define an existing fill or backfill stratum as either 'uncontrolled' or 'controlled' as this affects the Code-specified presumptive bearing value of the stratum material.

[12] The relatively new *GeoTech Tools* website (www.geotechtools.org, accessed 31 October 2014) contains an excellent summary of current ground modification/improvement technologies. Although this website was developed primarily for road-related applications, the technologies it contains are applicable to a much wider variety of structures and applications.

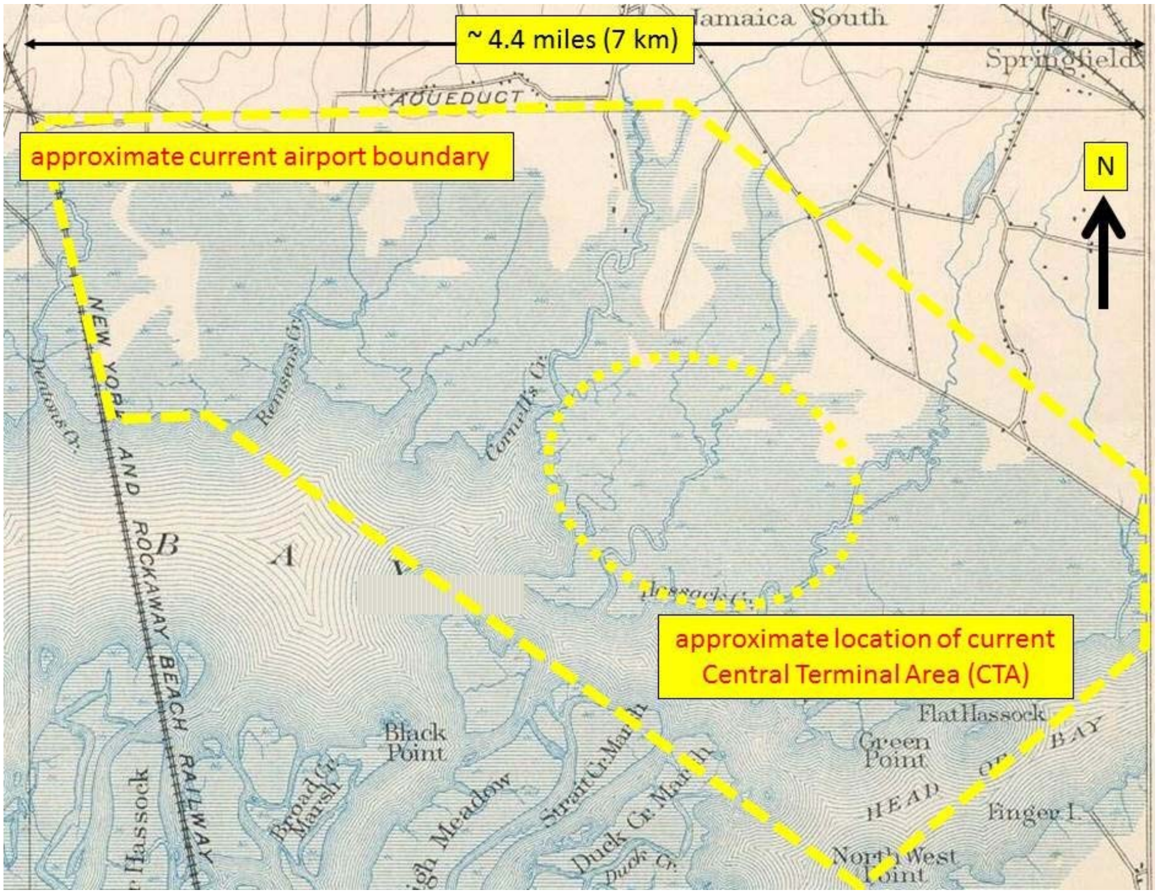

Figure 4. Portion of USGS[13] 1891 (1888-9 Data) Topographic Map Showing Current Significant JFKIA Geographical Features.

Regional Geology and Hydrogeology

Overview

The NYC metropolitan area arguably has the most diverse and complex geology of any major urban area in the world as a result of over a billion years of still-visible geological history that is still evolving due to ongoing soil deposition in many waterways (Horvath 2013). Recent primers on the subject can be found in USGS (2003), Bennington and Merguerian (2007), and Merguerian (2007).

Because of the temporal extent, complexity, and diversity of the geological record in this region, its complete interpretation is not a settled matter. In fact, certain aspects have undergone significant rethinking in recent decades and are still a work-in-progress as new investigations are made and ground-truth obtained. Nevertheless, the basic sequence of events is now well established and widely accepted.

It is of interest to note that much of the recent geological re-interpretation has been facilitated by engineered construction that has provided geologists with either direct physical access via open-cut excavations, shafts, and tunnels or soil and rock samples

[13] U.S. Geological Survey.

obtained in borings. Collectively, this has provided the ground-truth necessary to support much of this rethinking.

A summary of the key aspects of this geological history of relevance to foundation designers and constructors in the NYC metropolitan area can be found in Horvath (2013). The most significant outcomes of this history relevant to the focus of this paper are:

- Multiple episodes of tectonic-plate activity have shaped the underlying crystalline bedrock[14] regime both directly in terms of rock types, depth to rock, etc. and, more importantly in the case of JFKIA, indirectly in terms of the seismic potential resulting from the complex network of brittle faults, both known and presumed, throughout the region.

- A geologically-recent (Pleistocene Epoch[15]) cycle of glacial advances (glacials) and retreats (inter-glacials), all of which not only affected but, more importantly, terminated within various portions of the NYC metropolitan area. This glacial and inter-glacial activity is responsible for much of the soil and most of the geomorphology that existed prior to human modifications and alterations of the regional landscape in the last several hundred years. Of particular importance to the focus of this paper is the significant eustasy (sea-level change) associated with the alternating glacial and inter-glacial Pleistocene cycles combined with the isostasy (crustal vertical displacements) and eustasy of the subsequent Holocene Epoch that has resulted in the NYC metropolitan area in general, and JFKIA in particular, being a marine-coastal area not only in the present (see Figure 1) but at various times in the relatively recent past as well.

Structural Geology

Despite the overall complex geology of the NYC metropolitan area, the near-surface geology that directly controls key geotechnical issues as well as foundation design and construction at JFKIA is relatively simple and, as noted previously, remarkably consistent and uniform throughout most of the airport property. This latter aspect contrasts sharply with many other places in the NYC metropolitan area where geotechnically-significant changes in near-surface geology can occur just within a building's footprint or similar short horizontal distance.

To begin with, within the limits of JFKIA relatively ancient metamorphic crystalline bedrock is estimated to vary in depth in a more or less uniform, planar fashion from approximately 650 feet (200 m) to almost 1,000 feet (300 m) below sea level, with a NW-to-SE[16] dip (Daniels and Leo 1985, Buxton and Shernoff 1999). For reference purposes, the airport is approximately 13 feet (4 m) above current Mean Sea Level (MSL) and there is an average tidal range of about 5 feet (1.5 m) in adjacent Jamaica Bay.

Although crystalline bedrock does not directly influence or affect foundation design and construction at JFKIA it has an indirect effect by virtue of its influence on seismic

[14] This distinction of 'crystalline bedrock' is made given the significant differences in definition and interpretation of the more generic terms 'bedrock' or 'rock' between civil engineers and geologists as discussed in detail in Horvath (2013).

[15] North American naming nomenclature is used for all glacial and inter-glacial cycles mentioned in this paper.

[16] For simplicity throughout this paper and where the context is obvious, the standard compass orientations of North, South, East, and West will be denoted by the use of the capital letters N, S, E, and W respectively, either alone or in combination as appropriate.

design. Specifically, due to the ability of the significant thickness of overburden soils overlying bedrock to modify both the amplitude and frequency content of bedrock motions that are transmitted upward through the soil column during an earthquake, the depth to bedrock does impact the free-field ground-surface motions throughout the airport property. As will be seen, seismicity has become an important issue in recent decades at JFKIA, not only for the direct-shaking effect on foundations but, more significantly as it turns out, the potential for liquefaction.

Despite the fact that the underlying metamorphic bedrock is relatively ancient (estimated to have an age approaching one billion years in places), the natural soils overlying bedrock at JFKIA are considerably younger in geologic age. They can be divided into three major groupings as follows, from bottom to top (Buxton and Shernoff 1999; Moss 2013; Perlmutter and Geraghty 1963; Soren 1971, 1978):

- Several hundred feet (metres) of both coarse- and fine-grain soils from the Upper Cretaceous Period that extend to within approximately 300 feet (90 m) below MSL[17]. There is substantial physical evidence that during the Late Pliocene/Early Pleistocene Epoch an ancestral channel of the Hudson River incised a deep, roughly N-S trending valley in these strata, the thalweg of which underlies JFKIA as shown in a graphic on the cover page of this paper (Moss 2013, Soren 1978).

- Pleistocene Epoch deposits that extend from approximately 300 feet (90 m) below MSL to close to current MSL within Jamaica Bay as well as slightly above current MSL to form both the numerous islands of various size now or formerly located throughout the Bay and the upland ground surface west, north, and east of the Bay that comprises Long Island[18]. Within the JFKIA area, the most significant of these Pleistocene strata for the purposes of this paper is the uppermost stratum of micaceous coarse-grain outwash[19] from the most recent (Woodfordian) glacial cycle. This Woodfordian outwash, referred to as the Upper Glacial Aquifer in the various groundwater reports referenced above (e.g. Buxton and Shernoff 1999), extends to a depth of approximately 100 feet (30 m) below MSL throughout the entire airport. Not only does the Upper Glacial Aquifer serve as the bearing stratum for all deep foundations at JFKIA, as will be seen the gradation and consistency of soils comprising this stratum controls other geotechnical design aspects such as liquefaction for all major structures at JFKIA.

- The previously noted Holocene Epoch MTM deposits consisting of peat and/or organic clay. As can be seen in Figure 4, most of the current airport property, including all of the Central Terminal Area (CTA) for commercial passenger traffic, was formerly a tidal wetlands located along the northeast side of Jamaica Bay (refer to Figure 1 for an overall view). Note that the base map for Figure 4 reflects survey data obtained in 1888-9

[17] It is important to note that at the peak of the most recent Pleistocene glacial cycle (Late Wisconsinan a.k.a. Woodfordian, the latter term will be used in this paper) sea level is estimated to have been as much as 400+ feet (120+ m) lower than at present.

[18] Throughout this paper the place-name 'Long Island' is used in its strict geographical sense to include Kings, Queens, Nassau, and Suffolk counties. This usage should not be confused with the common colloquial, regional meaning that includes only the latter two suburban counties and omits Kings and Queens counties that are, respectively, the Boroughs of Brooklyn and Queens of the City of New York.

[19] The various Pleistocene terminal moraines that have been identified conclusively to date all lie several miles (kilometres) to the north of JFKIA as discussed in detail by Sanders and Merguerian (1995, 1998). However, it is of interest to note that there has been speculation in recent years that an as-yet-unconfirmed additional terminal moraine lies to the south of JFKIA in an area that is now covered by the Atlantic Ocean, thus making conclusive investigation difficult.

before any significant human development of the area was apparently undertaken and consequently shows the several tidal creeks, some of them connected to upland freshwater streams and ponds, that traversed the area in a nominally NE-to-SW flow direction. Decker (1946) reported that prior to construction of JFKIA the peat deposits, which would have comprised the surficial soils for portions of the wetlands that were at or above Mean High Water (MHW), ranged from 3 to 5 feet (1 to 1.5 m) in thickness. The organic clay, which Decker referred to variously as "Galveston clay" and "mud", ranged from 2 to 8 feet (0.6 to 2.4 m) in thickness and would have either underlain the peat within land areas or formed the bottom soils within tidal creeks, channels, and open-water areas within Jamaica Bay. Thus the aggregate thickness of the MTM soils prior to construction apparently ranged from approximately 5 to 13 feet (1.5 to 4 m).

Groundwater

Freshwater supplied from the ground for the entire gamut of human uses (personal, commercial, industrial, agricultural) has long been a key issue related to and interacting with the human settlement and development of Long Island. Although the earliest development of Long Island, which was concentrated in what is now part of NYC (Kings and Queens counties), did make some use of locally-constructed reservoirs and other surface bodies of water, the density of human development that occurred in the 20th century, especially in suburban Nassau and Suffolk counties, combined with the complex geology and inescapable fact that Long Island is surrounded entirely by saltwater bodies has made it literally a textbook case in groundwater (over)usage and contamination of different kinds (Kehew 2006).

Although the demand on the aquifers underlying the extreme western portions of Long Island has lessened in recent decades as Kings County (Borough of Brooklyn) and Queens County (Borough of Queens) have been increasingly connected to the NYC water-supply system (which is supplied almost exclusively by a complex system of reservoirs located north of the City), there is still significant demand from the remaining two suburban counties (Nassau and Suffolk). As a result, over the years there have been numerous groundwater-related studies performed at the Federal, State, and county levels. Some reports that include the area around JFKIA that have been published subsequent to construction of the airport are, in chronological order, Perlmutter and Geraghty (1963), Soren (1971, 1978), Buxton and Shernoff (1995), USGS (1997), Buxton and Shernoff (1999), and Cartwright (2002).

However, these and other studies have understandably focused on the deeper, initially-artesian, confined aquifers that are within the Cretaceous and older-Pleistocene strata discussed previously as these aquifers evolved through the course of the 20th century to be the primary sources of potable groundwater throughout Long Island. Thus while these confined aquifers were central to these various groundwater studies, geohydrological issues within these aquifers have no direct impact on foundation design and construction at JFKIA other than the fact that large-scale, regional changes in piezometric levels and/or groundwater density (the latter affected by salinity due to saltwater intrusion of formerly-freshwater aquifers) will affect the vertical effective stresses within these strata and could cause either regional subsidence or heave. Such considerations are beyond the scope of this paper and, as far as it is known to the writer, have never been studied for JFKIA.

Because of the deep-aquifer focus of these regional groundwater studies, relatively less attention has been paid in these studies to the shallow groundwater-table aquifer within the Upper Glacial Aquifer (Buxton and Shernoff 1999) that is the bearing stratum for all deep foundations installed at JFKIA. Nevertheless, there is some discussion of the Upper

Glacial Aquifer in these various references as part of an overall treatment of how groundwater levels and quality (including saltwater intrusion and human contamination) throughout the study areas have been affected by human development.

However, the most significant issue concerning groundwater conditions at JFKIA is that because most of the airport was created by landfilling within either an open body of water or the tidal zone of an adjacent land area, this obviously completely altered the regional hydrogeological setting and created a new, localized groundwater regime that is unique to the JFKIA property. This will be discussed further subsequently as part of the site-development history.

Seismology

Up until the final decades of the 20th century, the prevailing wisdom among geologists and civil engineers alike was that the potential for seismic activity of any significance to constructed facilities did not need to be considered when designing structures in the NYC metropolitan area. This was likely due, at least in part, to the fact that seismic activity of any significance had, for all practical purposes, been absent during the period of major growth and construction within the region that began in the late 19th century combined with the realization during the early decades of plate-tectonics research that this area is currently far removed from any current, active tectonic-plate boundaries[20]. It was only after extensive geological research in the last few decades, still a work-in-progress as noted previously and especially true for seismological aspects, that it was understood that the NYC metropolitan area had, in fact, been at the center of multiple episodes of tectonic-plate interactions in the form of collisions and separations in the geologic past, and that this activity had left numerous brittle faults of various size and significance (in terms of potential for future movement) throughout the region.

Unfortunately for geological and civil engineering analysis and design purposes, the explicit number and locations of these brittle faults are not known in areas such as Long Island where relatively thick deposits of soil cover the crystalline-bedrock surface and prevent documentation of fault traces. A clear example of this can be found in what is believed to have been the largest seismic event to affect the NYC metropolitan area since 1677, an estimated M = 5.25 event in 1884 that was epicentered just south of JFKIA (Sykes et al. 2008)[21]. Not only was the epicenter located within an open body of water (Atlantic Ocean) but there is estimated to be well over 1,000 feet (300 m) of soil overlying crystalline bedrock in that area which precludes linking a specific fault to this event.

This recently developed knowledge concerning the tectonic history of the NYC metropolitan area combined with careful study of the regional historical record (Sykes at al. 2008) plus a broader appreciation of East Coast seismicity in general has led to the recognition that the seismic potential throughout the NYC metropolitan area is significant enough for it to be considered as part of the design of most engineered structures. This is reflected in current hazard-mitigation plans prepared at both the state (New York State Multi-Hazard Mitigation Plan 2014) and local (New York City Hazard Mitigation Plan 2014) levels as well as incorporated in the NYC building code for several years now.

A detailed presentation and discussion of the earthquake-related sections of these hazard-mitigation plans is beyond the scope of this paper as this information is readily available on the Web (New York State Multi-Hazard Mitigation Plan/Section 3.7-

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[20] Recall that the concept of plate tectonics only received widespread recognition and acceptance in the final third of the 20th century.

[21] All earthquake magnitudes, M, reported and discussed in Sykes et al. (2008) are short-period body-wave magnitude, m_{bLg}.

Earthquakes (2014) and New York City Hazard Mitigation Plan/Section 3.9-Earthquakes (2014) respectively), as is the NYC building code. Only a synthesis and summary of these documents as they relate to seismic design at JFKIA is presented in this paper.

The current state-of-knowledge suggests that it is reasonable anywhere in the NYC metropolitan area to design for a seismic event in the M = 5-to-6 range with a very shallow focal depth and peak bedrock acceleration in the range of 0.14g to 0.15g. Note, however, that throughout the NYC metropolitan area the thickness of soil cover can vary significantly as can the nature of the soils that comprise this cover. The relevance of this is the well-known effect that soil cover can have on modifying bedrock motion so that the resulting free-field ground-surface motion is significantly different than the bedrock motion.

This issue is addressed in the New York City Hazard Mitigation Plan/Section 3.9-Earthquakes (2014) combined with the New York City Building Code/Chapter 18-Soils and Foundations (2014). These documents show the entire JFKIA area as having National Earthquake Hazard Reduction Program (NEHRP) Class D site conditions with a mandated default design value of adjusted (for site effects) value of the peak ground acceleration, PGA_M, = 0.24g which is more than a 50% increase over bedrock motion.

In addition to the obvious direct-shaking consequences of current seismic design throughout the NYC metropolitan area, an important consideration in many locales, including JFKIA, is the potential for liquefaction. In some ways, liquefaction is a more serious concern than direct shaking in an area such as NYC that has extensive land area with subsurface conditions that are potentially liquefiable combined with a utility infrastructure (especially water-supply and natural-gas lines) that has not been designed for seismic loading in any way. There is evidence that in the proper geological setting (i.e. relatively loose coarse-grain soils with a high groundwater table, conditions that are found widely throughout Long Island as well as elsewhere in the region), a M = 5-to-6 event could cause liquefaction. Thus it is no surprise that the current (2014) edition of the NYC building code mandates that a site-specific liquefaction assessment be performed for new structures under certain conditions that are discussed in detail later in this paper.

In conclusion, as will be discussed in the section on technical-needs assessment, the relatively recent recognition of the need to design for earthquakes in the NYC metropolitan area has had a significant effect on deep-foundation design at JFKIA.

Site-Development History

Pre-Airport

Available information from recent archaeological assessments (Kearns et al. 1991, Scharfenberger and Davis 2005) that geographically bracket the west and northeast borders of the JFKIA property respectively combined with examination of historical maps and newspaper articles dating back to the 1870s that were found as part of the research for this paper collectively indicate that human habitation and usage of natural resources within the area surrounding what is now JFKIA began with indigenous Native American peoples well before the first European colonization of the New York City metropolitan area in the 17th century CE (Common Era). Development of the area continued and expanded as a consequence of European colonization but was relatively limited until the late 19th century when a railroad line[22] was built along a nominally N-S alignment across Jamaica Bay as

[22] The current Howard Beach-JFK Airport station on the NYC subway system is a successor of this original rail line.

shown in Figure 5, just to the west of the current JFKIA property that is shown by red dashed lines in this figure.

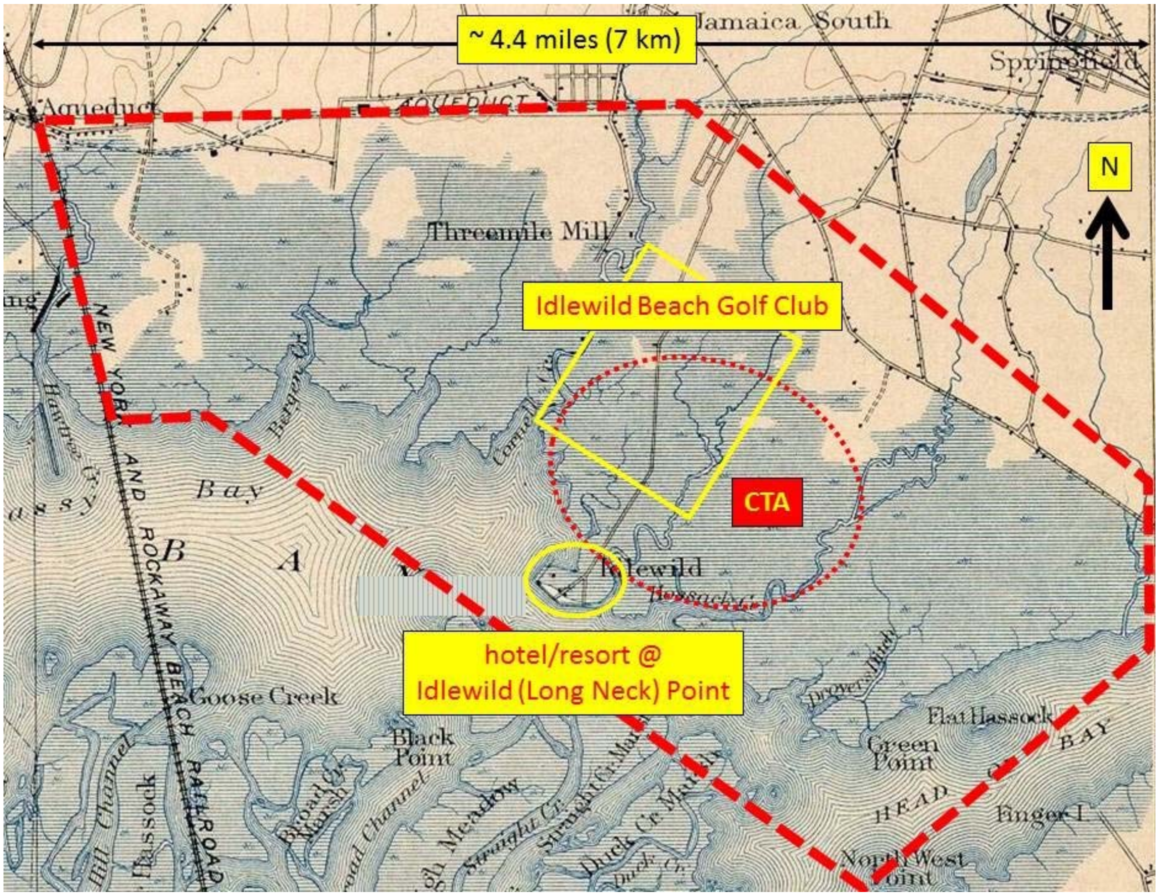

Figure 5. Portion of USGS 1898 (1897 Data) Topographic Map Showing Past and Current JFKIA Geographical Features.

That the future JFKIA area was largely bypassed by human development until the late 19th century is not surprising considering that most of the nominally land areas actually consisted of brackish-water tidal wetlands around the periphery of Jamaica Bay, with only isolated islands and peninsulas of permanently dry land as can be seen in Figures 4 and 5.

It appears that human development within the future JFKIA area began to increase toward the end of the 19th century. By comparing survey data obtained in 1888-9 (Figure 4) with that obtained less than a decade later in 1897 (Figure 5), it can be seen that modest development inroads had been made at several locations, especially within the area highlighted by the yellow oval in Figure 5 that is labeled "Idlewild" on the base map. The correct etymology of this place-name and when it first came to be used for this specific area are unknown to the writer at this time although there is no shortage of speculation in this matter[23]. In any event, because this development lies within the current footprint of JFKIA

[23] When the writer started working for the PANYNJ in June 1972, the anecdotal story told by long-time employees was that this area was once the 'playground' of the 'idle rich', hence the name. Other sources found on the Web during the course of research for this paper claim that the name derives from a Native American place-name for the area.

(just to the south of the most southerly CTA passenger terminals in an area occupied at present by aprons, taxiways, and Runway 13R-31L) it is of interest to discuss it in some detail.

To begin with, there is the issue of the correct place-name of the peninsula on which this development was sited. The name that appears most often in the late-19th century press was Long Neck Point[24], apparently related to the tidal creek (Long Neck Creek) that defined its eastern boundary (this creek bifurcated the current CTA). However, some maps from 1891 found during the course of research for this paper refer to it alternatively as Longneckers Point and Logneck (sic) Point, the latter likely a cartographer's error.

Curiously, contemporaneous USGS documents (the two versions of topographic maps used as the base maps for Figures 4 and 5 as well as a groundwater-related document (Veatch et al. 1906)) plus other maps and newspaper articles found during the course of research for this paper used a place name that is nowadays deemed extremely derogatory and unacceptable for use in any context. Consequently, this place name will not be mentioned in this report, even for or in its historical context (note that the writer edited out this place name on the base maps used for Figures 4 and 5). In any event, by some point in the early 20th century it appears that the place-name had morphed to Idlewild Point and that is the name that will be used in this paper.

Before delving into the history of human development at Idlewild Point, it is of interest to speculate as to the reason for its attraction for habitation given that it was in the proverbial 'middle of nowhere' as can be seen in Figure 5 (the road shown leading to Idlewild Point was a later addition as discussed subsequently). It appears the fact that this location was at the head of Broad Channel, at that time the largest natural channel in Jamaica Bay, and accessible by boat even at low tide (which rendered much of the Bay impassable tidal mud flats) was a significant factor in its development.

A fairly extensive discussion of the human development of Idlewild Point up to 1897[24] indicates that there may have been some type of structure built near there as early as the American War of Independence (Revolutionary War) in the late 18th century. What is known with more certainty is that there was a hotel of some sort that was built circa 1863 or perhaps sometime prior that became the Idlewild Club House of the Idlewild Club of Jamaica in that year. A map dated 1873 found during the course of research for this paper indicated a structure simply labeled "Club House" at Idlewild Point. Sometime later, after a bankruptcy sale in 1885[25], it was renamed the Idlewild Hotel.

Major changes to Idlewild Point began in the 1890s when Leon(h)ard Eppig, a Brooklyn brewery owner, purchased not only Idlewild Point but ultimately a 1.5-mile (2.4-km) long swath of the entire peninsula over which he built the first road access to Idlewild Point which had previously been accessible only via water. The route of the plank-road that was constructed can be seen in Figure 5 and traversed the heart of the current CTA. At the same time, substantial additional construction was undertaken that included a variety of structures, perhaps the most distinctive of which was a combination light house (electrically-lit), observatory, and enclosed water tower/tank that was 80 feet (24 m) high and no doubt visually distinctive (see Page iii of this paper) throughout the relatively flat marshlands of Jamaica Bay. There were also some substantial marine facilities that included a 4,000-foot (1,200-m) long timber bulkhead plus coal- and ice-handling facilities. All of this

[24] www.newspapers.com/image/50369763/. Accessed 11 November 2014.
[25] fultonhistory.com/Newspaper%2018/Hempstead%20NY%20Queens%20County%20Sentinel/Hempstead%20NY%20Queens%20County%20Sentinel%201884-1885/Hempstead%20NY%20Queens%20County%20Sentinel%201884-1885%20-%200240.pdf. Accessed 9 November 2014.

and more are discussed in some detail in the newspaper article cited in Footnote 24. Around this point in time the development apparently began to be referred to as Idlewild Park.

Freshwater was apparently provided by wells. Veatch et al. (1906) document a two-inch (50 mm) diameter well that was installed at Idlewild Point at some unspecified date. It was drilled to a depth of 200 feet (61 m) below ground surface (BGS), which was likely only several feet above MSL, and drew water from the Jameco Aquifer which is the oldest Pleistocene stratum in the region and considered hydrogeologically one with the underlying Magothy Aquifer of Upper Cretaceous age (Buxton and Shernoff 1999). This well was reported to have freshwater under flowing-artesian conditions at the time of the Veatch et al. report. Veatch et al. also document another well drilled somewhere in the vicinity of Hook Creek which is immediately to the southeast of JFKIA. This well was drilled to an almost identical depth (203 feet (62 m) BGS), also into the Jameco Aquifer and also exhibiting freshwater, flowing-artesian conditions at the time.

Moving into the 20th century, historical information is less detailed. References refer to this hotel variously as the Idlewild Hotel (circa 1900) and Hotel Idlewild On Jamaica Bay (circa early 1920s) in addition to the aforementioned Idlewild Park. As discussed subsequently, structures apparently existed as late as 1932 but whether or not they were still in operation at that time is unknown.

Circa 1921 a major transformation or expansion of Idlewild Park was envisaged in the form of "2,000 waterfront bungalow" sites for sale to the general public[26]. No conclusive information was found by the writer to be able to state with certainty whether or not such a development ever occurred but an aerial photograph from 1924 available at a City of New York website[27], a portion of which is shown in Figure 6 (note the scale and north arrow in the lower-right-hand corner of the figure), does not appear to indicate any level of development approaching 2,000 individual bungalows at that time. There does, however, appear to be a few buildings of modest size more or less fronting on Jamaica Bay which is toward the bottom of the figure. These buildings, and the well-defined linear form of the aforementioned timber bulkhead that is also quite apparent in this figure, are likely the remnants of Eppig's late-19th century Idlewild Park development efforts.

An observation relative to the base map used for Figure 6 is that it appears to have be taken at low tide. Extensive mud flats that appear at low tide are typical geomorphological features of marine tidal marshes in the NYC metropolitan area and they show up nicely as the extensive areas of lighter gray within the darker water areas.

The darker water areas are natural channels created by tidal currents as well as net seaward flow from the numerous tidal creeks that once flowed from upland freshwater sources in a general NE-to-SW direction in the area as shown in Figure 5. Such channels remained passable by shallow-draft boats even at low tide. The relatively wide channel toward the bottom of Figure 6 was the aforementioned head of Broad Channel and as noted previously was likely the feature that attracted human habitation of Idlewild Point at such an early date.

Another noteworthy feature in Figure 6 is the aforementioned Long Neck Creek which is the sinuous dark area of deeper water lying along the right (east) side of Idlewild Point in the figure.

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[26] query.nytimes.com/mem/archive-free/pdf?res=9805E5D61431EF33A25752C1A9649D946095D6CF. Accessed 26 August 2013.
[27] maps.nyc.gov/doitt/nycitymap/. Accessed 8 September 2013.

Figure 6. 1924 Aerial Photograph Showing Idlewild (Long Neck) Point.

Even though this large-scale bungalow project does not appear to have occurred it is clear that eventual development of most of the tidal wetlands surrounding Jamaica Bay within the area that would eventually become JFKIA was at least anticipated in the early 20th century. At least by 1923 a complete street grid with named and numbered streets appeared on at least one map found while preparing this paper[28]. Interestingly, the street grid did not extend to and include Idlewild Point although it did include most of the rest of the peninsula that terminated at the Point.

The apparent lack of follow-through on the planned Idlewild Park bungalow community is hardly surprising as several pieces of information found during the course of research for this paper suggest that not only did this extensive bungalow development never occur but the entire human use of Idlewild Point for recreational purposes collapsed and was ultimately abandoned. This was apparently due to extensive contamination of

[28] nycedges.blogspot.com/2011/01/islands-of-jamaica-bay-broad-channel.html. Accessed 8 September 2013.

Jamaica Bay by human activity, primarily the disposal of human and human-generated wastewater into the Bay with little or no treatment. It is known that swimming and fishing in the Bay were banned circa 1916[29] followed by the banning of commercial oyster harvesting, which was once a major commercial activity within the Bay, circa 1921[30]. There is additional anecdotal information from 1940 in the form of a quote from *Torreya*, the bimonthly newsletter of the then The Torrey Botanical Club (currently The Torrey Botanical Society), describing a field trip made by members in August 1940:

> *"...we changed to the bus from Jamaica and rode a short distance to 157th Street. This street is the road that crosses Idlewild golf course and on to Idlewild Point...Idlewild Point is a wide stretch of gravelly sand that extends half a mile out into Jamaica Bay. Years ago it was a small popular resort with hotel, cottages and bathing facilities, but is now deserted because of the polluted condition of the bay.*" - Volume 41, No. 3, May-June 1941[31]

In addition to the reference to contamination of Jamaica Bay and the implication that at least by 1940 the structures associated with the former hotel complex at the Point might have been demolished, there are other items of interest in this quotation. One is the comment that Idlewild Point was composed of "gravelly sand". The surface and near-surface Pleistocene soils found throughout this area tend to have a medium-to-fine gradation. Thus if the surficial soils at Idlewild Point were indeed "gravelly" it begs the question as to their origin, i.e. natural or human. It may well have been the latter as the late-19th century development undertaken by Eppig apparently involved the placement of "over 2,500 tons (2,270 Mg) of broken stone[24]" as part of the extensive timber bulkheading. This material would have remained, of course, after any timbers had rotted away.

Before returning to this quote, this discussion of the development history of Idlewild Point can be brought to a close by noting some items of relevance to this paper. The significance of this history for the present-day JFKIA is that there are quite possibly relict features of this prior construction now buried under airport property. While the above-ground portions of any structures were likely demolished prior to placing fill for the airport (as discussed subsequently, there are indications the major structures still existed as late as 1932), the timber piles that were driven for various structures (more than 3,000 were reportedly driven for the bulkhead alone[24]) almost certainly survive to this day as would areas covered by the substantial quantity of crushed rock that was placed. The buried timbers used for the plank-road may well exist also. These all represent obstructions that could serve as unexpected impediments to future construction, especially pile driving.

Moving on now to the "golf course" mentioned in the above quote, it was formally named the Idlewild Beach Golf Club and information found during the course of research for this paper indicates that it was located just to the north of Idlewild Point, within the general area of the yellow rectangle shown in Figure 5[32]. As can be seen in this figure, it appears that this golf course overlaps a significant portion of the current CTA. For reasons that are broadly similar to those involving development at Idlewild Point, it is of interest to discuss the history of this golf course to the extent possible using information found during the course of research for this paper.

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[29] www.nycgovparks.org/parks/B165/history. Accessed 8 September 2013.

[30] query.nytimes.com/mem/archive-free/pdf?res=9D05EED8103FE432A25753C3A9679C946095D6CF. Accessed 8 September 2013.

[31] archive.org/stream/torreya4041boni/torreya4041boni_djvu.txt. Accessed 26 August 2013.

[32] A circa-1940 commercial map that shows the location and extent of this golf course more explicitly can be found here: forgotten-ny.com/2000/04/ramblersville-queens/. Accessed 1 November 2014.

To begin with, it is unclear when this golf course first opened. The circa-1924 aerial photograph[26] of the area available on the Web from the City of New York is not definitive one way or the other. There are indications[33] it was initially planned and operated as a private nine-hole course. However, what is known with greater certainty is that by 1930 it was open to the general public[32,34], quite possibly the result of the onset of the Great Depression causing a strategic rethinking by the original (presumably private) developer(s). There were also plans at that time to expand it to 18 holes[32].

However, information published in 1932[35] is in partial conflict with this 1930 information as it indicates that only six holes of the course were then in operation (all located within the natural upland areas away from Jamaica Bay) although the remaining 12 holes were reportedly in various stages of completion at that time. Also noted with regard to construction of this golf course is an item of significant relevance to this paper:

> *"Millions of cubic yards of sand pumped from the bottom of the bay have raised the tide-washed land above the reach of the sea."*

Taken at face value, this indicates substantial filling had occurred in the early 1930s within the current CTA, roughly a decade before the substantial filling (discussed in the following section) that created the airport property. In addition, there are indications that the filling done for this golf course was graded to create the usual rolling topography typical of such facilities. How this fill compares in terms of gradation to that used for the airport is not known, nor is the effect on any variable topography (the airport filling was, overall, more or less level). In any event, indications are that this golf course was still in existence in 1940[30] and 1941[36] and may well have existed until airport construction began in 1942.

Another item of interest from this same 1932 newspaper article[34] about the golf course is a casual reference that indicates the major structures related to the hotel complex at Idlewild Point were still standing at that time:

> *"...the same as the feeling when walking over Idlewild and looking out across the bay...Only the old Eppig House, one-time meeting place of the prominent Germans of Brooklyn with its water tower and old-fashioned hotel look, crops out on the horizon to make one realize that he's at Idlewild..."*

It is interesting to see that "Eppig House" was apparently at least a colloquial name for the Idlewild Hotel or may even have been its official name after the complex was extensively rebuilt and revitalized in the late 1890s[24].

To conclude this discussion of pre-airport history, information was found during the course of research for this paper suggesting that Idlewild Point was also considered in the early decades of the 20th century as the site for a completely different usage, the location of a coastal-artillery battery[37] for the defense of New York Harbor against a potential enemy fleet. For whatever reason(s) this was never pursued.

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[33] fultonhistory.com/Newspaper%205/Brooklyn%20NY%20Daily%20Eagle/Brooklyn%20NY%20Daily%20Eagle%201930%20Grayscale/Brooklyn%20NY%20Daily%20Eagle%201930%20a%20Grayscale%20-%200128.pdf. Accessed 1 November 2014.
[34] bklyn.newspapers.com/image/59863077/. Accessed 1 November 2014.
[35] www.newspapers.com/image/59865681/. Accessed 1 November 2014.
[36] www.newspapers.com/image/52805035/. Accessed 1 November 2014.
[37] www.metropostcard.com/metropcbloga10.html. Accessed 8 September 2013.

Until the 1930s, the primary commercial airport serving NYC was not located within the City itself or even within the State of New York but was the original[38] Newark Airport southwest of NYC in New Jersey. The first significant commercial airport located within the City was LaGuardia Airport (LGA)[39] in northwestern Queens, built on and around what was originally Sanford Point and the site of a pre-existing private flying-field and amusement park before that[40]. Substantial landfilling within adjacent Bowery Bay and Flushing Bay also occurred as part of LGA construction. Construction and initial operation of LGA was entirely a City of New York undertaking as the PANYNJ did not take over operation of LGA until 1947, almost a decade after flight operations began there in 1939.

Apparently no sooner had LGA opened than it was judged to be at capacity. As a result, plans were initiated by the City of New York in 1941 to create an additional, larger airport within City limits. Initial plans called for it to be a joint project between the City of New York and Federal government, with the former paying for all structures and the latter for the filling and site grading[41].

In reading the quoted remarks of then-NYC mayor Fiorello LaGuardia in the newspaper article cited in Footnote 41, it is clear that his vision for the new airport included air freight (a first for a NYC airport at the time), which remains a major component of JFKIA operations to the present, as well as "enormous" growth in post-World War Two trans-Atlantic air travel. The latter not only turned out to be true but remarkable given that he said this before the U.S. entered that war. Two of LaGuardia's prophecies that did not come true were that the airport would be in service before the war ended and that it would be used by the U.S. Navy for training purposes. To the best of the writer's knowledge JFKIA has never hosted any long-term, permanent military function.

The site chosen for this new airport, which was originally to be named Idlewild Airport, was within the current footprint of JFKIA. Original plans called for a facility occupying only 500 acres (200 ha) which is only about one-tenth the area covered by the present airport and actually smaller than the current size of LGA. This was increased almost immediately to 1,200 acres (485 ha) which is still only about one-quarter of the current size of JFKIA. The new Idlewild Airport was to be located roughly at the site of the golf course shown in Figure 5 (the existence of the Idlewild Beach Golf Club was noted in the newspaper article cited in Footnote 41 but the implication was that this would not be any impediment to developing the airport) and overlapping today's CTA.

A map from circa 1940 found during the course of research for this paper also shows a private flying-field named Queens County Airport in this general location. It is of interest to note that other maps found during the course of research for this paper show other private flying-fields located within the footprint of what would eventually become JFKIA. One was Sunrise Airport (circa 1940, located within the extreme western portion of what would become JFKIA, near the long-term parking lots and current Howard Beach-JFK Airport subway station) and Jamaica Bay Airport (circa 1930, located within the extreme southeastern portion of what would become JFKIA and abutting Nassau County).

As an aside, it appears that in the early days of aviation when there was considerably less regulation of flying it was not uncommon for privately-owned flying-fields

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[38] The Newark Airport one sees today as a passenger has existed since the early 1970s and bears no resemblance to the original facility that was much smaller in size and occupied the northeast portion of the current facility.

[39] The original name was New York Municipal Airport-LaGuardia Field.

[40] www.airfields-freeman.com/NY/Airfields_NY_NY_Queens.htm. Accessed 9 September 2013.

[41] www.newspapers.com/image/52805035/. Accessed 1 November 2014.

to be established at will that were often little more than an unpaved landing strip. In any event, other than these existing flying-fields plus the possibly-abandoned hotel complex at Idlewild Point and Idlewild Beach Golf Club discussed previously the area that would eventually become JFKIA was either open water within Jamaica Bay or tidal wetlands with the isolated fishing shack or cottage as shown in the undated photograph in Figure 7.

Figure 7. Typical Surface Conditions Within Marine-Tidal-Marsh Areas Prior to Construction of JFKIA [photo credit: Downer, Green and Carrillo].

Construction of Idlewild Airport began in April 1942, in the early months of U.S. involvement in World War Two. The most significant aspect of the general earthwork to develop the site was the placement of hydraulic fill to create a new land mass that was adequately above sea level (the official elevation of the current airport from an aviation perspective is +13 feet (+4 m) Above Mean Seal Level (AMSL)). All fill soil was dredged[42]

[42] As a bit of civil engineering trivia, Decker (1946) notes that one of the dredges used on the project was the *Nebraska* which had just finished working on the well-known Fort Peck Dam project on the Missouri River in the State of Montana. This dam is apparently still the largest in the world constructed by hydraulic filling. This dredge was disassembled; shipped by rail to the Port of Albany, NY on the Hudson River where it was reassembled; then towed almost 200 miles (300 km) to the project site in Jamaica Bay.

from the adjacent portion of Jamaica Bay called Grassy Bay (the location can be seen in the center-left portion of Figure 5) and then pumped as a watery slurry (an 85%:15% ratio of water to soil particles according to Decker (1946)) for distances of up to 3 miles (5 km).

Considering that approximately 66,000,000 cubic yards (50,000,000 m^3) of hydraulic fill was eventually placed to create the original airport (Decker 1946), the gradation of the fill soil turned out to be remarkably uniform: a brown, micaceous sand of typically medium-to-fine gradation, that would colloquially be referred to as a 'beach sand' in the NYC metropolitan area. All fill material is the most-recent (Woodfordian) glacial outwash from the uppermost portions of the Upper Glacial Aquifer that underlies the entire Jamaica Bay area below the Holocene organic soils.

Decker (1946) goes into considerable detail about many details related to the site development of Idlewild Airport, most of which need not be repeated here for the purposes of this paper. However, of relevance is that in the early years at least the overall project was apparently very much a constantly evolving work-in-progress with regard to overall airport size as well as the general location and orientation of the CTA and runways. The final design as reported by Decker called for six pairs of parallel runways radiating from the CTA in a pinwheel layout so that any three adjacent runways could be used simultaneously for departures and the three mirror-image runways for arrivals. In addition, early in construction (1943) the facility was renamed from Idlewild Airport to Maj. Gen. Alexander E. Anderson Airport[43].

No photographs have been found to date by the writer that show the airport under construction (wartime restrictions may have played a role in taking aerial photos in particular). However, a stand-in to illustrate hydraulic filling within Jamaica Bay is Figure 8. This is a photograph that shows essentially identical hydraulic filling in progress for the embankment portions of the Cross Bay Boulevard road project which would date this photo from the early 1920s. The view in this photo is to the north and the aforementioned rail line built in the late 19th century is the dark line going from upper-left to lower-right.

Some scale of what is shown in Figure 8 can be deduced by noting that the distance portrayed in the vertical direction is of the order of 4 miles (6 km). The light-colored material along the left side of the photo is already-placed hydraulic-fill sand for Cross Bay Boulevard. The former Howard Beach railroad station (current Howard Beach-JFK Airport subway station) is located at the top of the photo. The area that would eventually become JFKIA is the land and water area in the upper-center and upper-right portions of this photo. Grassy Bay (the portion of Jamaica Bay that would be the source of hydraulic fill for the airport) lies in the upper-center of the photo.

Although the nature and gradation of the hydraulic-fill sand used to create the landfill for the airport is uniform, the fill thickness above the pre-existing Holocene MTM stratum is quite variable. It ranges from zero in the more northerly portions of the airport property that were already permanently above water prior to construction to perhaps 20 feet (6 m) or more in southerly portions of the site that were within Jamaica Bay.

Of course final fill thicknesses would have reflected primary consolidation plus any secondary (creep) compression of the MTM organic soils which Decker (1946) said compressed up to 50% in places (Figure 9 is an undated photo but shows a typical settlement plate being placed on the surface of the MTM stratum prior to filling). Decker also states that only three to four months was required for consolidation of the MTM soils to be completed with no further settlement although he presented no plots or hard data to support this or to show that no secondary (creep) compression was ongoing at the time the paper was written.

[43] query.nytimes.com/mem/archive/pdf?res=F10816FE3F59147B93C7AB178DD85F478485F9. Accessed 26 August 2013.

Figure 8. Circa Early-1920s Aerial Photograph Showing Hydraulic Filling for Embankment Portions of Cross Bay Boulevard Under Construction.

Figure 9. Settlement Plate Placement Prior to Hydraulic Filling.

The information provided by Decker (1946) concerning thicknesses of the hydraulic fill and pre-existing Holocene MTM soils is consistent with the writer's experiences that are primarily within the current CTA. Current subsurface conditions within the CTA typically consist of a surficial hydraulic-fill sand stratum of the order of 10 to 15 feet (3 to 4 m) thick underlain by Holocene organic soils (peat and clay) that are typically no more than about 10 feet (3 m) thick and more often of the order of about 5 feet (1.5 m) thick. This is followed by the natural Pleistocene outwash sands of the Upper Glacial Aquifer that extend to the maximum depths of typical geotechnical borings and CPT soundings (generally about 100 feet (30 m) BGS).

Before continuing with the airport development history, it is of interest to digress to discuss a significant geotechnical consequence of the construction as it relates to hydrogeology. A geotechnically-significant outcome of this site development is that the local hydrogeological regime was fundamentally and permanently altered so that a special, localized groundwater regime exists within the JFKIA property. The Upper Glacial Aquifer throughout the natural upland areas around the airport is naturally a groundwater-table aquifer (Buxton and Shernoff 1999) although where it was locally covered by the Holocene MTM stratum around the fringes of Jamaica Bay it would have been a confined aquifer under flowing-artesian conditions due to the relatively impervious nature of the MTM soils. With the placement of the hydraulic-fill stratum on top of the MTM soils, the fill stratum became the groundwater-table aquifer within the new land mass that was created (and will be referred to hereinafter in this paper as the Hydraulic Fill Aquifer) with the Upper Glacial Aquifer a confined aquifer, except at the periphery of the airport property where the Holocene MTM stratum naturally thinned and disappeared. In such areas the Upper Glacial Aquifer plus any additional sand fill that might have been placed to raise grades to desired levels remained the groundwater-table aquifer.

The relevance of this to geotechnical and foundation engineering at JFKIA is that in any hydrogeological setting where two aquifers (one the groundwater-table and the other confined) are separated by a well-defined aquitard (aquiclude), such as the Holocene MTM stratum in this case, the default condition is that the two aquifers will have different piezometric levels, i.e. hydrostatic groundwater conditions do not exist throughout the entire subsurface profile. Typically and in the absence of any human modification the underlying confined aquifer is under artesian conditions, often flowing-artesian conditions.

In the specific case of JFKIA, over the years the writer has seen varying and conflicting data from project-specific open-well piezometers (observation wells) installed to measure piezometric levels in both the Hydraulic Fill Aquifer and Upper Glacial Aquifer to define the piezometric level in each. In some cases the piezometric levels appear to be the same while in other cases they differ slightly. It may well be that both conclusions are correct depending on where specifically one is within JFKIA because of the relatively thin nature of the Holocene MTM stratum combined with the thousands of penetrations of this stratum by foundation piling that allows substantial aquifer communication between the strata. This communication could allow the aquifers to equilibrate piezometrically, at least in some portions of the airport.

In any event, because the difference in piezometric levels between these two aquifers tends to be small at best it is common on projects at JFKIA to assume that hydrostatic groundwater conditions defined by the groundwater level exist within the depths that are relevant to foundation and other geotechnical assessments at the airport (approximately 100 feet (30 m) maximum BGS as noted previously). Within the CTA at least groundwater is typically less than 10 feet (3 m) BGS with an elevation that is slightly above Mean High Water (MHW) in Jamaica Bay.

Returning to the site-development history, Figure 10 is an aerial photograph taken in June 1947 showing the original airport filling essentially completed (additional filling within Jamaica Bay to extend runways would occur in later years) but the airport not yet in operation. This view is to the southeast with Jamaica Bay to the right and the Atlantic Ocean at the top. The original pinwheel runway layout mentioned previously is clearly visible and with so much exposed area of relatively clean, fine sand it is clear why Decker (1946) noted significant construction problems with blowing sand[44] as well as why at the time the site was apparently referred colloquially to as the 'Jamaica Bay Sahara'.

Figure 10. Aerial View of Airport on 5 June 1947 [photo credit: Fairchild Aerial Surveys].

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[44] In subsequent years, other projects within the NYC metropolitan area that involved filling or creating large land areas using essentially-identical clean, fine sand dredged from within Lower New York Bay (an area also underlain by Woodfordian outwash sands) suffered identical problems with blowing sand. However, in the case of these later projects there were adjacent developed areas (roadways and buildings) so the negative impact was much greater and more problematic than during construction of JFKIA.

On 1 June 1947, the City of New York leased the still-unfinished airport to the PANYNJ which opened the airport for commercial service in July 1948. The PANYNJ formally renamed the facility New York International Airport-Anderson Field[45] although photographs of the original terminal buildings only show the 'New York International' part of the name displayed. However, the writer can attest from first-hand experience that until the airport was renamed JFKIA in 1963 it was, at least within the NYC metropolitan area, universally referred to colloquially as 'Idlewild Airport' or simply 'Idlewild'. Indeed, from the day commercial operations began in 1948 until renaming in 1963 the official industry codes (technically known as Location Identifiers) for the airport implied the original Idlewild name (FAA LID and IATA: *IDL*; ICAO: *KIDL*).

The airport was officially renamed John F. Kennedy International Airport in December 1963 and the aviation Location Identifiers changed accordingly to *JFK* and *KJFK* that remain to the present. However, throughout the NYC metropolitan area at least the renamed airport is universally referred to colloquially as 'Kennedy Airport' or simply 'Kennedy'.

Additional building and rebuilding has occurred since the initial construction of the airport. In recent decades at least some of this expansion and reconstruction has been episodic in the form of themed programs, at least within the CTA. These will be discussed as appropriate in later sections of this paper.

Summary of Key Points

The technical needs with respect to foundation design and construction at JFKIA are defined by:

- <u>nature</u> in the form of site conditions which, in the case of JFKIA, have been significantly altered by human activity during the course of initial airport construction and

- <u>humans</u> which, in the case of JFKIA, has been and is primarily the vision and policies of the airport lessee and operator, the PANYNJ, functioning as the de-facto owner although the airlines have had and continue to have varying degrees of input and control for various passenger and cargo terminals as well as hanger and maintenance facilities.

With regard to site conditions, historically the primary technical needs were defined solely by the development history of JFKIA and the fact that within the depth that governs foundation design (approximately 100 feet (30 m) BGS) most of the airport property is underlain by the following three-component system of Quaternary soils (listed in descending order):

- <u>Holocene hydraulic-fill sand</u> that, while generally of good quality from the perspective of particle-size distribution (micaceous medium-to-fine sand with a trace of silt) and potential geotechnical load-bearing, is classified as uncontrolled fill due to the nature of its placement. There is no indication that any attempt was made to densify or otherwise improve the entire thickness of fill soil after placement other than whatever densification occurred incidentally during site grading and intentionally during creation of pavement systems. Thus this stratum is considered nominally unsuitable as a bearing stratum for structures per applicable building codes. This Holocene fill stratum is a

[45] 'International' was reportedly included in the formal airport name as 'New York Airport' sounded too much like 'Newark Airport' when spoken and for obvious reasons it was necessary to avoid any confusion between the two, especially in operational communications with aircraft.

groundwater-table aquifer, with groundwater levels throughout the CTA at least relatively shallow (generally less than 10 feet (3 m) BGS) and only a few feet (metres) above MHW in Jamaica Bay.

- Holocene marine-tidal-marsh organic soils (peat and/or organic clay deposited in a brackish-water environment) that are also considered nominally unsuitable as a bearing stratum per applicable building codes.

- Pleistocene outwash sands from the most-recent (Woodfordian/Late Wisconsinan) glacial cycle that is predominantly a quartz sand with varying gradation (coarser) and density (denser) with depth. Because this stratum was the source material for the surficial Holocene hydraulic-fill stratum, the upper portion of the Pleistocene outwash and the Holocene fill stratum are visually and gradationally identical. One of the signature mineralogical features of this soil is its significant mica content, especially muscovite which creates a visually-distinctive appearance for this soil. The mica content can skew particle-size/grain-size distributions curves developed using traditional sieve analysis that are, by default, based on the assumption that the specific gravity of solids, G_s, of all the particles is the same. The specific gravity of micas (biotite and muscovite) has some range but tends to be close to 3 whereas the specific gravity of quartz is nominally 2.65. Correction for this is typically not done in practice. In addition, unpublished data available to the writer for high-quality triaxial tests performed in the early 1970s on these soils suggest that the presence of mica apparently results in values of Mohr-Coulomb friction angle, ϕ, under critical-state (constant-volume) conditions that are slightly higher than might be expected for pure quartz sands of similar gradation. Hydrogeologically, the Pleistocene outwash stratum is known as the Upper Glacial Aquifer (UGA) and is now a confined aquifer within most of the airport except toward the N-NE limits of the airport property where both the Holocene fill and underlying organics thin out and disappear and the Upper Glacial Aquifer becomes the surficial soil stratum and groundwater-table aquifer. However, within most of the airport property where the Holocene MTM stratum exists this stratum is an imperfect confining layer (aquitard/aquiclude) for the UGA with some aquifer communication occurring between the Holocene sand fill and Pleistocene sand so that any head difference between the two strata is small to non-existent. This is likely due to the relatively modest thickness of the Holocene MTM as well as the fact that it has been penetrated in thousands of places by foundation piling driven through it.

To transition to the technical discussions in the following section of this paper, Figure 11 illustrates this stratigraphy for the typical conditions found within the CTA using both a conventional boring with Standard Penetration Test (SPT) sampling as well as well as a CPT sounding[46] that were performed in relatively close proximity to facilitate direct

[46] As noted previously and discussed in detail later in this paper, the PANYNJ was a relatively early (for the NYC metropolitan area) proponent of using of CPT (and later CPTu) soundings to supplement and complement traditional borings and concomitant SPT sampling in geological settings conducive to CPT/CPTu usage. The NYC metropolitan area has not seen extensive use of CPT/CPTu soundings as the glacially-impacted subsurface conditions found throughout the region often preclude use of this in-situ testing tool. However, the subsurface conditions at JFKIA are textbook conditions of where CPT/CPTu soundings should arguably be the site-characterization tool of choice although this has not occurred to date.

comparison. Although the particular results shown in this figure were obtained in late 1988[47] they are representative of conditions found to the present.

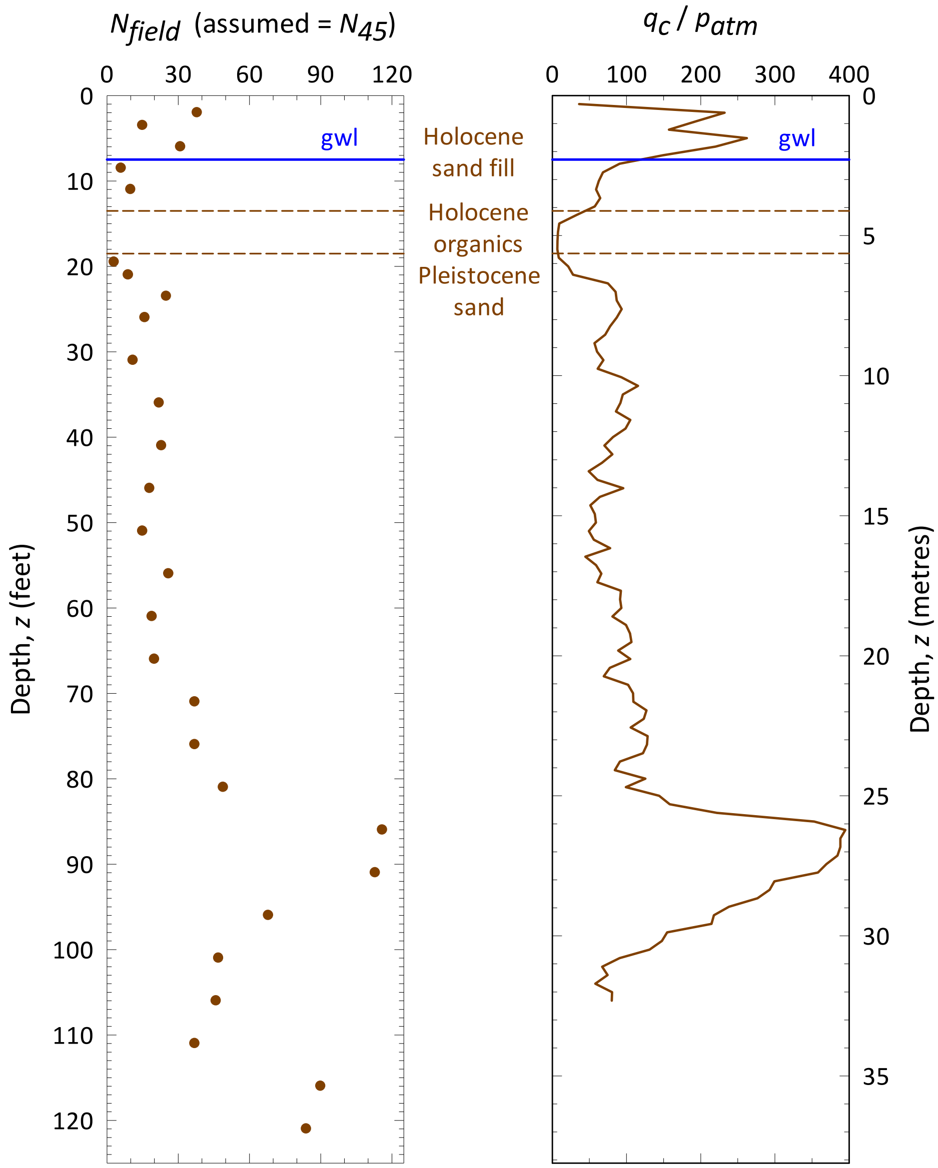

Figure 11. Typical Subsurface Conditions within CTA (1988 PANYNJ Field Data).

The assumption made in Figure 11 that the field (raw) SPT *N*-values (N_{field}) were obtained using a hammer-drive system with 45% efficiency (implied by the $N_{field} = N_{45}$ indicated in the figure) was not based on explicit energy measurements but the writer's assessment of the type of system used for this particular boring (traditional donut hammer with rope wrapped around the cat head of the drill rig, the predominant drive system used

[47] The cone sounding shown in this figure is actually a composite, the arithmetic average of two closely-spaced, almost-identical soundings that were both located close enough to the borehole shown so that the stratigraphy in each can be considered directly comparable. Later sections of this paper will use these two CPT soundings separately.

in the NYC metropolitan area during and before the timeframe this boring was drilled) and empirical correlations appearing in numerous publications (e.g. Kulhawy and Mayne 1990).

From the perspective of building codes as well as the state-of-practice during the life of the airport, the Holocene sand fill and organic soils of the Holocene MTM strata are individually and collectively deemed to be unsuitable for load bearing with the net result that shallow foundations are not a technically viable alternative for any significant structure at JFKIA. By its nature, a large commercial airport such as JFKIA has many building and transportation structures that are relatively sensitive to total and differential settlements which has meant and continues to mean that all such structures must be supported on deep foundations bearing within the Pleistocene sand stratum.

As will be seen in the next section dealing with how these technical needs have been assessed and turned into practical designs, although neither the subsurface conditions nor the types of structures supported at JFKIA have changed materially since airport construction began in 1942 there are two noteworthy factors that have had a profound influence on the evolution of deep-foundation design there:

- At least since the early 1970s, a significant portion of the building and rebuilding at JFKIA has been episodic in the form of a publicized (at least within the PANYNJ) campaign or program, e.g. *JFK 2000* in the latter years of the 20th century which was, as the name implies, an effort to modernize certain airport facilities for the new millennium. The significance of this is that it has usually meant that there was ample PANYNJ funding available to conduct test-pile programs whose intent was to both fine-tune and improve current design alternatives as well as evaluate new alternatives with the ultimate goal of improving the efficiency of deep-foundation designs for local subsurface conditions and minimizing the cost of deep-foundation systems as measured in dollars per kip (or kilonewton) of axial-compressive resistance. As a result of these test campaigns (the writer is aware of three that have occurred beginning in and since 1972, there may have been more), the state of deep-foundation practice at JFKIA has tended to change in well-defined, discrete increments as opposed to continuously.

- Although nature (in the form of subsurface conditions) has not changed since the basic airport earthwork were completed in the 1940s, the human interpretation and understanding of nature, specifically and especially the potential for seismic activity in the New York City metropolitan area and its effect on the Pleistocene Upper Glacial Aquifer that comprises the bearing stratum for all deep foundations at JFKIA, has undergone significant change in the 70-plus years since construction of JFKIA began.

TECHNICAL NEEDS: ASSESSMENT

Introduction

This section of the paper deals with how the unique site- and project-specific set of foundation needs defined by the interaction between natural surface plus subsurface conditions and the development plans of humans at the JFKIA site have been interpreted by design professionals and translated into foundation designs. It should be no surprise that this interpretation and concomitant translation has changed in many ways over time...some drastic and revolutionary, others more subtle and evolutionary...given that the more than seven decades of foundation history at JFKIA coincided with a period of time in human history where there has been exponential growth in technology[48].

Given that technological changes have occurred in many and varied ways, the presentation and discussion of this section of the paper is organized into several topics, each of which the writer feels has been a distinct area of technological change that has impacted foundation design and construction at JFKIA in some meaningful way.

Environmental Awareness

It is of overall interest to note that broad, legislated environmental awareness and sensitivity simply did not exist when JFKIA was first conceived and built. At that time, wetlands were called swamps and were viewed as insect-infested areas best drained or, better yet, filled over to make 'productive' use of them. There was a similar attitude towards placing fill within a body of open water to create made-land for likewise 'productive' use. Furthermore, soils underlying areas of open water such as Jamaica Bay were viewed solely as a non-renewal resource to be mined for purposes such as landfilling.

Of course it is now recognized that wetlands and adjacent bodies of water are productive resources in their own right as essential components of an overall ecosystem. That Jamaica Bay and its remaining wetland areas are now part of the Gateway National Recreation Area and Jamaica Bay Wildlife Refuge only emphasizes this fact.

It is also of interest to note that the negative environmental impact that the construction of JFKIA had on the overall Jamaica Bay ecosystem was not a one-time thing. More than 70 years after construction of JFKIA began it continues to negatively impact the Bay's ecosystem in several ways, chiefly because of the deeper-than-natural bathymetry left in Grassy Bay due to dredging for the hydraulic-fill source material coupled with precipitation runoff from Runway 4L (original Runway 4) in particular which had been extended farther into Jamaica Bay subsequent to the original airport construction[49]. These long-term, chronic conditions are in addition to several episodic events involving spillage or leakage of aviation fuel within airport property. Given the relatively high permeability of the soils that comprise the Holocene hydraulic-fill stratum that is the groundwater-table aquifer (Hydraulic Fill Aquifer) underlying the airport property, the potential for fuel infiltration through the vadose zone to groundwater level and the subsequent relatively rapid migration of a plume of fuel is significant.

In conclusion, it appears safe to say that approval to build JFKIA where it is and in the manner it was constructed would likely not be granted in today's world. However, this is a moot point as the airport is already built. But it does mean that any foundation-related activities performed at the airport are now under various types of environmental scrutiny that did not exist in the past.

Computational Tools

Arguably the most basic, universal change in technology that has affected engineered construction has been the calculation hardware available to design professionals and contractors alike. The sliderule was the norm when JFKIA was first being

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[48] Compare this 70-year timespan that straddles the 20th and 21st centuries CE and its concomitant change in technology to the bell tower of the cathedral at the Piazza del Miracoli in Pisa, Italy (more commonly and colloquially known today as the Leaning Tower of Pisa) where construction techniques during the 12th through 14th centuries CE were relatively little changed over the almost 200 years it took to build this structure (Burland et al. 2009).

[49] library.fws.gov/pubs5/web_link/text/jb_form.htm

designed in the early 1940s and this continued for a few more decades until the evolution and miniaturization of electronics and associated technology such as microprocessors progressed to the point of what can be called routine practicality-of-use. In terms of the impact this had on the calculation tools available to those involved in engineered construction, this evolution took two distinct paths.

The one that had the more immediate impact but has progressively become less significant over time was the handheld calculator that emerged in the early 1970s although the initial versions were not only computationally primitive by today's standards but also relatively expensive for an individual to purchase[50]. It was well into the 1970s before calculator technology advanced and prices dropped to the point that the average design professional was expected to own one as simply a basic personal tool of their profession. This has continued to the present although much of the work performed originally using calculators has been taken over by digital computers. This is because when first introduced calculators were simply used to replace sliderules for solving the types of analysis and design methodologies that existed in the pre-computer age.

The other path, which was slower to develop and evolve but has had a much more significant and lasting impact, was the aforementioned digital computer that debuted originally in the form of large mainframe systems that required a dedicated computer center and knowledgeable operating staff. Such centers began to appear in universities and other large organizations by the 1960s[51].

However, for the typical civil engineer in private practice such centers were only accessible by either visiting them in person or, more commonly as the 1970s and 1980s progressed, accessing them via a remote terminal and landline telephone connection. This was the state-of-practice until well into the 1980s.

Although the personal computer (PC) had debuted some years earlier, it was not until the mid- to late-1980s that the computational power of the PC had increased to the point where it contained the critical mass of computational capability that could reasonably operate software of use to civil engineers. PC technology has continued to evolve in terms of hardware/devices and software to the present.

The development and evolution of these two broad categories of computational tools (handheld calculators and digital computers) available to design professionals has had other, collateral technological impact on geotechnical and foundation engineering due to the overall miniaturization of electronic devices in general and microprocessors in particular. In particular, this miniaturization has allowed these devices to become much more transportable in addition to smaller in size. These collateral impacts, including things such as cellular-telephone technology, have most affected how technical needs are fulfilled during construction and will be addressed further subsequently in that section of this paper.

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[50] The writer's first Texas Instruments calculator that was purchased in the late 1972-early 1973 timeframe only had basic arithmetic functions (addition, subtraction, multiplication, and division plus a few others) and cost US$125...almost three days' salary for the writer at the time...which was considered a relatively low, heavily-discounted price in the very price-competitive New York City marketplace. According to the U.S. Bureau of Labor Statistics, as of November 2014 that US$125 would be equivalent to approximately US$700 which is more than the cost of any number of microprocessor-based devices with drastically more computational capability.

[51] This webpage (www.columbia.edu/cu/computinghistory/36091.html, accessed 4 November 2014) provides a nostalgic look at the IBM System/360 installation that the writer used at Columbia University in the City of New York beginning in the late 1960s.

Geology

The writer has long felt that geology is an essential, indispensable tool in geotechnical and foundation engineering practice that is both underappreciated and underrated by many, arguably more than ever (Horvath 2013). While geological conditions per se may change little, if at all, in the human perception of time, the interpretation of these conditions as a result of continuous research has been nothing short of revolutionary even within the human perception of time. For example, when JFKIA was first constructed in the 1940s the now-fundamental concept of plate tectonics was not universally accepted or even fully understood by its proponents. Thus the impact of geology on geotechnical engineering practice, while always there, has increased significantly in recent decades.

The role played by geology can be both site-specific as well as regional. As a specific, relevant example of the site-specific utility of geology, consider that when using a relatively typical boring and sampling protocol for a proposed building (e.g. as specified in the City of New York building code) that subsurface soils are sampled at the rate of about one part per million (1:1,000,000), with any type of laboratory testing performed at an even larger ratio. Given that the subsurface materials encountered in geotechnical investigations were placed either by nature or undocumented/uncontrolled prior human activity, this sampling and testing frequency is much less than that for manufactured materials used in engineered construction such as steel and Portland-cement concrete (PCC).

Appropriate and relevant geological information can usually help remedy this situation in two basic ways:

- by visually filling in the physical gaps between samples as an outcome of knowing the depositional and other geomorphological history of a site and

- aiding in the geological interpretation of basic geotechnical tests such as the Standard Penetration Test (SPT) as illustrated by Moss (2012).

With specific regard to JFKIA, the numerous technical papers and reports cited earlier in this paper under the discussion of regional geology and hydrogeology (most of which were published well after initial airport construction) contain significant information about conditions within the JFKIA boundaries. This information is useful for understanding the several hundred feet (metres) of soil stratigraphy that extends beneath the typical depth of conventional foundation-related borings at the site (of the order of 100 feet (30 m) BGS as noted previously).

However, in recent decades the most useful geology-related contribution to geotechnical engineering practice in general and foundation engineering at JFKIA in particular has been the tectonic (literally and figuratively) paradigm shift in the understanding and appreciation of seismic potential in the NYC metropolitan area. It is now understood that a persistent, recurrent level of moderate seismic activity is part of the regional fabric for reasons adequately explained by plate tectonics and concomitant continental drift (Sykes et al 2008). This new-found recognition of seismic potential has by no means been limited to the NYC metropolitan area as the aforementioned emergence and acceptance of plate tectonics as a fundamental mechanism of Earth geology has led to a wholesale reassessment of seismic potential worldwide in general and along the East Coast of the U.S. in particular. A direct, foundation-related consequence of this is that it is now standard practice to consider lateral loading on foundations on a routine basis.

An indirect, corollary outcome to this new-found appreciation of the seismic potential in the NYC metropolitan area has been the recognition and understanding of liquefaction. Although this phenomenon has always been a secondary effect of earthquakes under certain geological and hydrogeological conditions, it was not recognized and understood in a widespread, meaningful way by geotechnical engineers until after two seminal seismic events that occurred within three months of each other in Anchorage, Alaska and Niigata, Japan in 1964, more than 20 years after construction of JFKIA began.

As it turns out, the issue of liquefaction potential has had much greater and more significant impact on foundation design at JFKIA compared to the lateral forces due to direct shaking. With reference to Figure 11, in the earlier decades at JFKIA when timber piles were used almost exclusively they would sometimes fetch-up, i.e. reach acceptable levels of axial-compressive geotechnical resistance per code-based protocols in effect at the time[52], within the first 10 feet (3 m) into the Pleistocene-sand bearing stratum.

Based on the current state-of-knowledge of NYC regional seismicity combined with the current state-of-practice for liquefaction assessment using SPT- and CPT-based analysis methodologies (which are discussed in considerable detail later in this paper), the prevailing professional opinion that has evolved in recent decades is that there is potential for liquefaction to occur within the upper several tens of feet (metres) of the Pleistocene-sand bearing stratum at JFKIA under earthquake magnitudes with a recurrence interval deemed reasonable for design of most structures (typically m_b[53] in the range of 5 to 6). The last such event in the region occurred in 1884 and is believed to have been epicentered in the Atlantic Ocean just south of JFKIA (Sykes et al. 2008).

The potential for liquefaction and its concomitant effect on the geotechnical capacity of deep foundations currently dictates that all new deep foundations installed at JFKIA extend beyond minimum depths BGS that typically exceed by a substantial margin the depths of driven piling installed earlier in airport history. While this should, in principle, provide for adequate foundation support of recently constructed structures built to such standards it begs the question of performance and survivability of older structures of all types that are supported on piles founded significantly or even entirely within the potential liquefiable zones.

Site Characterization

General Comments

The first time the writer saw the term 'site characterization' used in a geotechnical-engineering context several decades ago the immediate thought was "Why and why now?". Why was a new term needed at this point in time for something so basic that geotechnical engineers already did it instinctively as a natural, logical part of any project? Indeed, in many respects the same basic subsurface-exploration and soil-sampling technology centered around using the Standard Penetration Test (SPT) and the Standard Split Spoon that would have been available to civil engineers in the early 1940s still forms the backbone of site exploration to this day[54]. Nevertheless, time has proven the term site characterization to be durable so that it is now part of the routine lexicon.

That having been said, there is no denying the fact that the advances in site characterization in the decades JFKIA has been in existence have been nothing short of revolutionary. These advances have been not only been in terms of hardware for field exploration and sampling, laboratory testing, and, especially, field (in-situ) testing but also

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[52] Typically the *Engineering News* dynamic formula.
[53] Body magnitude.
[54] This is not something that the profession should necessarily be proud about as noted by Mayne (2012).

in the impact on all aspects of geotechnical analysis as a result of interpretative algorithms to transform raw in-situ testing data into fundamental soil properties. Indeed, the writer's former Manhattan College research project titled Integrated Site Characterization and Foundation Analysis that formed the basis for much of the analytical work reported in this paper owed its very existence to the proliferation of interpreted outcomes from in-situ testing that were developed in the latter decades of the 20th century and continue to be upgraded and developed in the 21st century.

Unfortunately, there is emerging evidence (with which the writer concurs completely) that the enormous strides in the overall site-characterization process, especially in-situ testing, have not permeated either education or routine practice to the extent they not only could but arguably should (Mayne 2012). In fact, there are indications that the state-of-practice with regard to at least some aspects of site characterization may have actually regressed in recent decades (DeGroot 2013). This, then, raises the obvious questions as to what advances in site characterization may have been and/or be of potential use at JFKIA and which of these, if any, may actually have seen use.

With reference to Figure 11 and the discussion up to this point, it is clear that the Pleistocene stratum of coarse-grain soil has always dominated foundation design at JFKIA and, more recently, has been the focus of concerns about potential liquefaction. Given the historical difficulty with relatively-undisturbed sampling and subsequent laboratory testing of coarse-grain soils, it is equally clear that in-situ testing and soil properties such as relative density and, more recently, stress history that can be inferred from in-situ test results are the aspects of site-characterization development of greatest relevance at JFKIA.

In-Situ Testing

As noted in the preceding section, in-situ testing is arguably the single most significant technological development related to site characterization that has occurred in recent decades although as Mayne (2012) points out that it has not been exploited in either education or practice to the extent it could or should. However, as discussed subsequently some of the newer developments in-situ testing has been utilized to some extent at JFKIA, especially since the late 1980s.

Before discussing specific in-situ testing devices in the following sections, it is useful to summarize the distinctly different ways in which in-situ test data can be used in practice as this can influence the selection or rejection of specific devices both in general as well as on a given project. To begin with, when an in-situ testing device is activated by advancing it into the ground or, alternatively, activating some portion of it while the overall device is otherwise stationary at some depth in the ground, one or more physical parameters are measured and recorded. Unlike traditional laboratory testing protocols that are typically crafted to more or less directly measure some material property or properties (e.g. stiffness, strength, permeability), the parameters measured with an in-situ testing device are typically something unique to the device and, in some cases, have no apparent relationship to fundamental soil properties. A well-known example of this is the field (raw) N-value, N_{field}, measured as a result of advancing the Standard Split Spoon into the ground during performance of the SPT. In and of itself, N_{field}, a dimensionless integer, has no obvious correlation with any fundamental soil property. This, then, defines the issue of how the parameters measured during performance of an in-situ test are used in some engineering analysis or design process, e.g. calculating the allowable geotechnical bearing pressure of a spread footing.

Experience indicates that two broadly different approaches have been and are used in practice:

- The in-situ test parameter or parameters can be input directly into some empirical relationship to produce a desired end result, a technique that is especially popular when the measured parameter(s) is/are something abstract and non-intuitive. The classical tripartite relationship between SPT *N*-value, allowable footing bearing pressure, and an implied 1 inch (25 mm) of settlement, which dates back to the original Terzaghi-Peck version of 1948 (Terzaghi and Peck 1967), is a well-known example of such an approach.

- The in-situ test parameter can be related to some fundamental soil property and then that property used with some theoretical solution, e.g. from or based on the theory of linear elasticity.

In the latter case, how the correlation between in-situ test parameter and soil property is achieved can vary in two general ways. When there is no obvious or intuitive connection (e.g. between CPT tip resistance, q_c, and relative density, D_r, of a coarse-grain soil) then an empirical relationship must be established using calibration-chamber and/or other testing. In other cases, the correlation is more intuitively obvious, e.g. interpretation of the pressure-volume curve for a pressuremeter (PMT) test for either stiffness modulus or shear strength.

Experience indicates that either approach can produce reliable results provided that the user understands the limitations of the knowledge-base on which a given methodology was developed. This means the scope of the database for an empirical relationship or the assumptions made for a theoretical derivation need to be clearly understood.

The writer has a personal bias toward and preference for correlating in-situ test results with fundamental soil properties then using these properties in theoretically-based solutions. This is because all steps in the process are more transparent and open to objective assessment compared to using a methodology that is basically a 'black box' based totally on empiricism. In particular, it is not always straightforward to know or understand the limitations of the database used to create the latter methodologies.

Standard Penetration Test (SPT)

Considering first the SPT, the oldest type of penetrometer in the ever-enlarging panoply of in-situ testing, the primary outcome of site-characterization research has been to illustrate and define the influence of various process variables (type of hammer-drive system, type and length of drill rods, etc.) on the measured field *N*-values (N_{field}). Kulhawy and Mayne (1990) provide a good summary of the key issues involved and the outcomes of research that have shown that the Standard Penetration Test is a remarkably non-standard when applied in the real world in routine practice.

In the writer's opinion, the most important SPT process variable is relative driving efficiency (typically expressed as a percentage between 0% and 100%) of the hammer-drive system, i.e. what percentage of the theoretical energy-per-blow (4,200 inch-pounds (475 J)) is actually delivered to the drive-head at the top of the string of drill rods to which the Standard Split Spoon is attached.

This driving efficiency is largely a function of the mechanics of the overall hammer-drive system used. There are strong correlations between common hammer-drive systems and ranges of efficiency which means that, as a minimum, the type of hammer-drive system should always be noted on the field boring logs by the boring inspector and carried over to the final boring logs prepared for any reports, contract documents, etc. This is something that was not done consistently historically and, in the writer's experience, still is not done routinely in general U.S. practice to the present. How consistently this is and has been done for borings drilled at JFKIA is not known to the writer.

The 'gold standard' with regard to SPT energy measurements is to obtain them explicitly using hardware manufactured specifically for this purpose, e.g. the *SPT Analyzer* from Pile Dynamics, Inc. Whether this has ever been done for borings drilled at JFKIA is not known to the writer. However, it is of interest and relevance to note that on projects where such measurements have been made the correlation between actual, measured efficiencies and efficiencies inferred based solely on the type of hammer system used is not perfect. For example, Gibbens and Briaud (1994) found that the actual average driving efficiency for several borings was 53% while empirical correlations suggest that it should have been 60%. The conclusion is that the empirical correlations between SPT efficiency and hammer systems should always be used with some caution but they are much better than doing nothing.

In any event, the issue of SPT driving efficiency is now considered to be an important one in practice for evaluating a wide variety of soil properties as well as the direct determination of both deep-foundation axial capacity and, especially, the potential for liquefaction. The current state-of-practice typically calls for normalizing N_{field} to, as a minimum, N_{60} if not $(N_1)_{60}$. This has been done by the writer for previously published research into driven-pile capacities at JFKIA (Horvath 2002) that is summarized later in this paper.

Quasi-Static ('Dutch') Cone Penetrometer Test (CPT)

Overview

Recent decades have seen a proliferation of in-situ testing devices in the broad family of penetrometers that are based on the common, core principle of advancing a metal rod with some type of shaped tip, often conical, into the ground using a downward-acting axial force applied concentrically to the top of the rod. This force is delivered to the top of the rod by either pushing (either manually or mechanically) or impact-driving in a manner similar to the aforementioned SPT (which is actually the oldest member of the penetrometer family). As a minimum, the tip resistance to rod penetration is measured in some fashion, i.e. mechanically or electrically.

Unfortunately, the writer has learned first-hand that there is conflicting and overlapping terminology used for these different types of non-SPT penetrometers, some of which come in variants in terms of not only their physical dimensions but the physical parameters they measure, that is exacerbated by the fact that not all penetrometers have been formally recognized and standardized by ASTM. In addition, different practice areas in civil engineering, i.e. foundation engineering and pavement design, and different geographical regions of the world have seemingly developed different devices with apparently minimal or no interaction and technology transfer between or among the different groups. Consequently, the net result is that there is some ambiguity in practice as to what is meant by a 'cone penetrometer' or 'cone penetration test'. Further complicating the terminology issue is the fact that the terms 'direct push' and 'direct-push system' seem to be used synonymously in at least some technical and geographical markets as an alternative term to 'cone penetration test'.

The only cone-type penetrometer that appears to be relevant for use at JFKIA for geotechnical and foundation engineering purposes and thus discussed in this paper is the classical device with a 60° apex angle at the tip that was developed originally in The Netherlands in the 1930s (Mayne 2007, Robertson and Cabal 2012). This 'true' cone penetrometer, which is the only penetrometer to which the acronym CPT should be applied, initially measured only tip resistance, q_c,[55], mechanically and saw increasing worldwide recognition, acceptance, and use (with ongoing improvements and embellishments that continue to the present) after World War Two.

When the writer first learned about this type of cone penetrometer decades ago it was still being referred to colloquially, in the U.S. at least, as the Dutch Cone in deference to its origins or, less commonly, as the Begemann Cone in recognition of H. K. S. Begemann who, in the 1960s, was instrumental in developing the friction sleeve that measured the parameter f_s to complement tip-resistance measurements. However, these terms appear to have since disappeared from routine usage and been replaced by cone penetration (or penetrometer) test or, more commonly, its acronym, CPT.

In recent years, the writer has come to appreciate that tip area, typically expressed in square centimetres[56], is a useful (but not unique) metric for differentiating between the various conical penetrometers. The technical relevance and importance of tip area is that the ability of conical penetrometers in general to sense changes in tip resistance is a function of tip diameter and, therefore, area. Nowadays, the CPT usually has a tip area of 10 cm^2 although in the early days of electronic CPTs tip areas of 15 cm^2 were apparently not uncommon, presumably a pragmatic necessity for accommodating the mensuration components of the day. Interestingly, the 15 cm^2 cone appears to be making somewhat of a comeback, apparently because it is a physically more-robust device in certain testing environments.

It is relevant to note that the acronym CPT is sometimes used in a broad, generic context (as it has up to this point in this paper) to include devices of this basic type that measure other parameters in addition to the basic q_c and f_s such as the most-common evolutionary improvement, the piezocone (CPTu), and a more-recent development, the seismic piezocone, sCPTu (SCPTu). This is because both the original CPT and CPTu/sCPTu provide the same basic information of tip resistance and sleeve friction and thus allow calculation of the dimensionless friction ratio, $f_s./q_c$, that is variously referred to using the notation R_f or FR and is sometimes expressed as a percentage. However, in the interest of technical accuracy in this paper the term CPT will be used only to mean the basic cone without pore-pressure capability. The basic mechanical-measurement version of the CPT is covered by ASTM Standard D3441 and the later, now-ubiquitous electronic-measurement version is covered by ASTM Standard D5778.

As noted previously, historically the CPT/CPTu/sCPTu has not been used widely in the NYC metropolitan area. The reasons for this are likely many and varied but certainly subsurface conditions are one significant technical factor. Given the landscape-changing impact that Pleistocene continental glaciation has had on the region, the widespread presence of gravel as well as cobbles and boulders[57] is a practical constraint to CPT usage in

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[55] There is a trend in recent years to report (sometimes exclusively) and use the corrected tip resistance, $q_t.$, (= q_c - u where u = the pore pressure measured at the common u_2 location on the device). However, the traditional uncorrected tip resistance, q_c, will be used primarily throughout this paper not only for historical purposes but also because the corrected tip resistance cannot be determined for older devices that did not measure pore pressure at all or did not measure it in the u_2 location behind the tip.

[56] This length unit is retained here for its past historical and current colloquial usage in this context even though the use of centimetre is deprecated in the SI system of metric units that is used as the secondary system of units throughout this paper.

[57] These terms are used in the context of the Unified Soil Classification System that defines 'soil' as all particles 3 inches (75 mm) or less in diameter with larger particles defined as 'rock fragments' with the further sub-divisions of 'cobbles' (3-12"/75-300 mm) and 'boulders' (>12"/300 mm).

at least some situations where it might be of value. However, portions of the region such as JFKIA that lie south of the known terminal moraines and are thus relatively devoid of larger granular soil particles as well as rock fragments are certainly prime candidates for CPT/CPTu/sCPTu usage as are the many land and marine areas where relatively soft fine-grain soils dominate the subsurface geology above bedrock.

Within this context of historical usage, it is of interest to note that the PANYNJ Engineering Department deserves credit for being relatively innovative in terms of local usage of cone penetrometers for geotechnical investigations. The writer is aware of CPT soundings conducted at JFKIA on behalf of the PANYNJ as early as 1988. An electrical cone with one-foot (300 mm) data-sampling intervals was used and it may have been an early version with a 15-cm^2 tip area (copies of the sounding logs provided to the writer by the PANYNJ years ago do not indicate the tip area of the cone).

In any event, in the writer's opinion the greatest site-characterization benefit of performing CPT/CPTu/sCPTu soundings in any type of soil is the ability to benefit from decades of worldwide research into developing theoretical and empirical algebraic relationships based on and around CPT/CPTu/sCPTu data to estimate a wide variety of soil properties. The calculated outcomes from these relationships can be used not only to complement, supplement, and replace conventional laboratory testing but to estimate soil properties in situations where laboratory testing is, for all practical purposes, impossible. This includes fine-grain soils in deep-water marine applications as well as coarse-grain soils in all applications.

It is this latter case that has intrigued the writer for decades now as the potential to develop a wide range of usable soil properties for coarse-grain soils on a routine basis using common investigatory tools could revolutionize geotechnical and foundation engineering practice as it would allow the routine use of theoretical and potentially more-accurate solutions. Consequently, beginning in the late 1980s and continuing to the present the writer has had a research interest in first developing and then refining on an ongoing basis a comprehensive, yet computationally simple (i.e. solvable manually using a hand-held calculator if desired although automation of the process using a computer greatly speeds up computation time when there are many pieces of input data), solution algorithm for coarse-grain soil. Use of this algorithm has been illustrated for a variety of practice-oriented applications such as shallow-foundation settlement and bearing capacity and the axial-compressive geotechnical ultimate resistance (capacity) of driven piles.

The Appendix of this paper summarizes the writer's published work in this regard as well as formally presents for the first time the latest update (Version 3.1) of the basic site-characterization algorithm developed by the writer. This algorithm makes use of several of the aforementioned empirical relationships developed by others. However, the unique aspect of this algorithm is that the writer has combined these relationships in what is believed to be a novel manner that allows a wide variety of soil properties to be estimated. Furthermore, this algorithm can be used even when only older CPT (as opposed to CPTu/sCPTu) data are available. This is significant as many of the current empirical relationships for soil properties require pore-pressure data from a CPTu or sCPTu.

As an additional practical consideration, this algorithm has intentionally been structured with the engineering practitioner in mind so that it can be used even if the only available field data are SPT *N*-values. Published relationships to convert SPT *N*-values to pseudo-q_c values allow for its use on the broadest range of projects possible as well for using data from the past. Of course it should be recognized that using only SPT *N*-values produces far fewer data points as a function of depth compared to even older CPT soundings, with concomitant inherently greater uncertainty in terms of soil-property variation with depth.

As noted previously, the unique element of the writer's algorithm for estimating soil properties for coarse-grain soil is not the empirical and theoretical relationships per se that it uses but the manner in which these algebraic relationships are interconnected to produce a wide range of outcomes, some of which such as overconsolidation ratio, *OCR*, would be otherwise difficult to come by using single, stand-alone relationships. However, the empirical relationships used by the writer as well as others can be used individually to estimate specific soil properties.

A discussion of all of these empirical relationships is beyond the scope of this paper. However, there are two specific sources that will be mentioned as material from each of them will be used later in this paper. First and foremost is the well-known work of Prof. Paul W. Mayne whose efforts to develop empirical relationships for soil properties based on in-situ testing devices in general dates back to Kulhawy and Mayne (1990). Mayne (2006, 2007, 2012, 2014) collectively provide a recent and comprehensive treatment of the subject with additional related material in Mayne et al. (2009).

The other resource is the work of Dr. Peter K. Robertson that is summarized in Robertson and Cabal (2012). This work also forms the basis and backbone for a commercially-available computer program named *CPeT-IT* (pronounced 'see petite') that is marketed and sold by GeoLogismiki[58]. The writer owns and has used this software[59] for site characterization at JFKIA. Some of the results are presented in this paper.

Applications at JFKIA

To show the utility and potential for using CPT and even SPT data for comprehensive site characterization in coarse-grain soil, some examples are presented in this section. Both the writer's proprietary[60] algorithm as well as selected alternative methodologies found in published literature will be presented.

The writer has long used JFKIA field data (borings, CPT soundings, test-pile load tests), provided to the writer years ago by the PANYNJ for academic research and educational purposes, to evaluate the writer's CPT-based analysis algorithm in general and its application to driven piles in particular. The writer's previously published academic research reports and conference papers itemized in the Appendix contain plots of the relevant soil properties produced as calculated outcomes from this algorithm.

At various times subsequent to these publications the writer has made revisions to this site-characterization algorithm to take advantage of new developments. Consequently, it is of interest to revisit some of this earlier published work using the current version (3.1) of the algorithm, as implemented in the writer's proprietary computer program *HINT*, as these results have not previously been published. Only selective plotted results will be presented here that focus on several parameters of use for deep-foundation axial geotechnical ultimate-resistance calculation as well as liquefaction assessment.

This paper makes use of one boring (No. 3-256) and two CPT soundings (Nos. CTP-2 and CTP-5) that were performed in 1988 for a test-pile program associated with one of the

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[58] www.geologismiki.gr. Accessed 17 November 2014.
[59] Version 1.7.6.42
[60] The writer's analytical algorithm is proprietary only in the sense that it is the result of the writer's original scholarly research. The actual mechanics of the algorithm have been published several times in no-cost venues over the years as detailed in the Appendix and thus can be implemented by anyone and at no cost without restriction or restraint. However, the computer program *HINT* that the writer uses to implement this algorithm is not and never has been available.

PANYNJ's facilities-construction campaigns named *JFK 2000*[61]. Although information concerning the absolute and relative locations of these three explorations was not provided to the writer, they were reportedly performed within the CTA and in sufficiently close proximity to each other so that they can be interpreted as having the same surface elevation and reflecting the same subsurface stratigraphy.

Before presenting various interpreted soil properties it is useful to look at plots of basic data obtained in both the boring and CPT soundings. To begin with, Figure 12 shows three types of SPT *N*-values as a function of depth BGS for this boring:

- N_{field} (which, as discussed previously, is assumed to be N_{45} based on the type of hammer-drive system used)
- $N_{60} = (ER_{field}/ER_{60}) \cdot N_{field} = (45/60) \cdot N_{field}$
- $(N_1)_{60} = C_N \cdot N_{60}$.

Because $(N_1)_{60}$ figures prominently in a wide variety of geotechnical calculations these days, it is relevant to note that it is not a unique quantity for a given value of N_{60} (which is unique for a given value of N_{field} assuming that the efficiency or energy ratio, *ER*, of the hammer-drive system used to obtain N_{field} is known). This is because over the years various researchers have suggested different empirical equations for the dimensionless correction factor, C_N, that is, as seen above, necessary to calculate $(N_1)_{60}$. Kulhawy and Mayne (1990) noted seven such equations and in the 20-plus years since then more have been added to the list[62]. However, as Kulhawy and Mayne note the primary difference between and among the different empirical normalization equations is within relatively shallow depths where $C_N > 1$. At greater depths where $C_N < 1$ the differences are relatively insignificant. As it turns out, only these greater depths are of interest at JFKIA so selecting a normalization relationship is not as critical.

Nonetheless, the writer chose the following relationship for C_N based on the observation that it appears to be the one of choice in several recent publications dealing with liquefaction which is not only a common analytical use of $(N_1)_{60}$ but of particular relevance to this paper:

$$C_N = 0.77 \cdot \log_{10}\left(\frac{40}{\sigma'_{vo}}\right) < 2 \qquad \textbf{(1)}$$

where σ'_{vo} is the vertical effective overburden stress and has units of kips/ft^2.[63]

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[61] The field data for this boring plus a composite of these two CPT soundings are what is shown in Figure 11.

[62] it is of historical interest to note that early efforts to adjust (normalize) SPT *N*-values to a constant stress level did not always use the reference-stress level (i.e. stress at which $C_N = 1$) of 1 atmosphere that is, by definition, common to all relationships for $(N_1)_{60}$. For example, the earliest attempt to normalize *N*-values that the writer is familiar with is the work of Bazaraa (1967) who used a vertical effective overburden stress level of 1.5 kips/ft^2 (71.9 kPa) at which to set $C_N = 1$. This is approximately 75% of atmospheric pressure.

[63] Note in Equation 1 that $C_N = 1$ when $\sigma'_{vo} = 2.011$ kips/ft^2 (96.33 kPa) which is actually slightly less than normal atmospheric pressure (2.116 kips/ft^2 = 101.3 kPa).

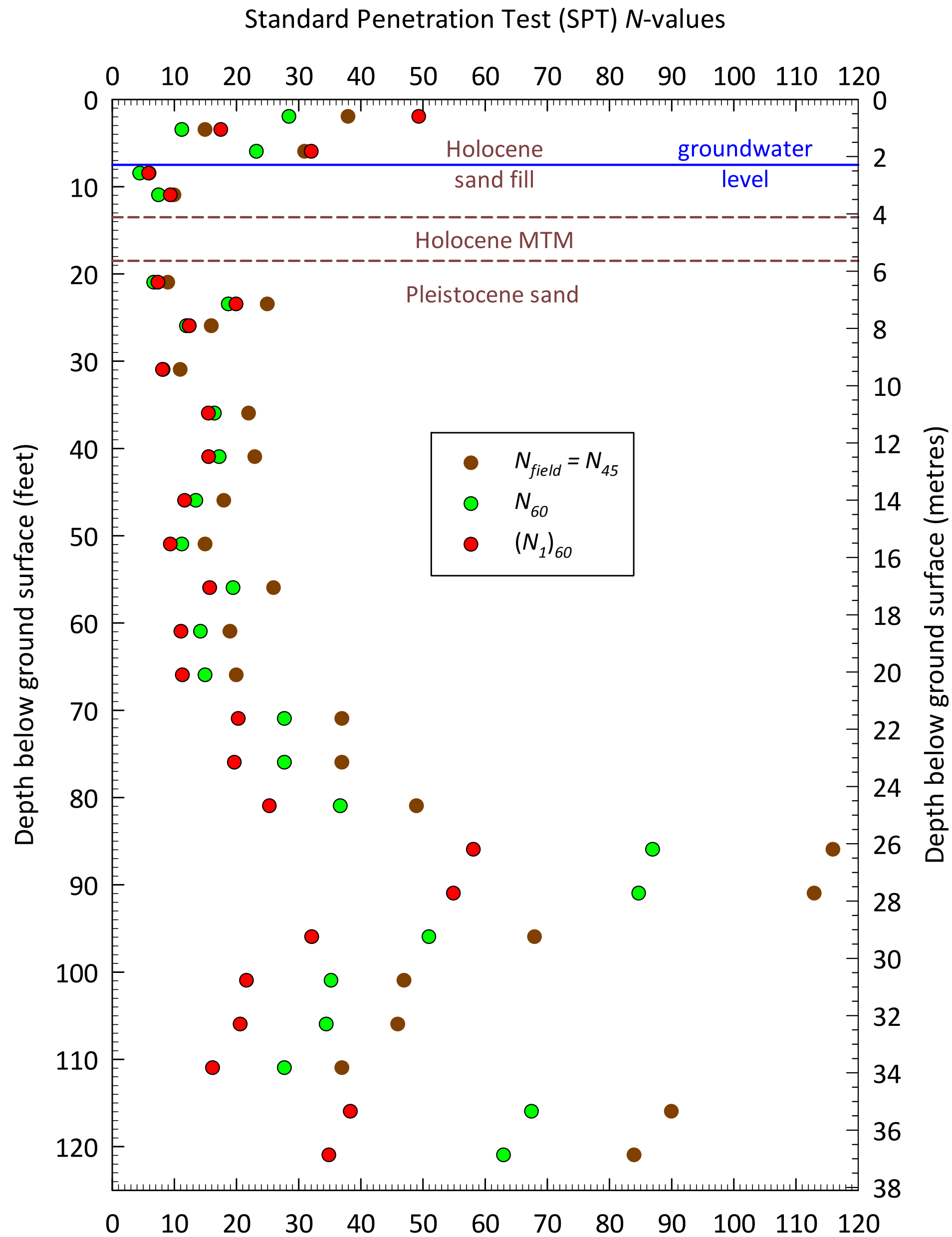

Figure 12. SPT *N*-values for Boring No. 3-256 within CTA.

Figure 13 shows the non-dimensionalized (to atmospheric pressure) uncorrected tip resistance, q_c, for the two CPT soundings as a function of depth BGS. Also shown are the pseudo-q_c values calculated using the N_{60} values from the boring and the relationship given in Horvath (2000a, 2002, 2011).

As noted previously, in recent years it has become more common to plot q_t, the tip resistance corrected for porewater pressures acting on the annulus behind the tip just below the friction sleeve. However, necessary information to do this for the soundings shown in this figure was not available to the writer. However, the correction is typically small in magnitude and relatively insignificant for terrestrial soundings in coarse-grain soil so this deficiency is not believed to significantly impact or influence the results.

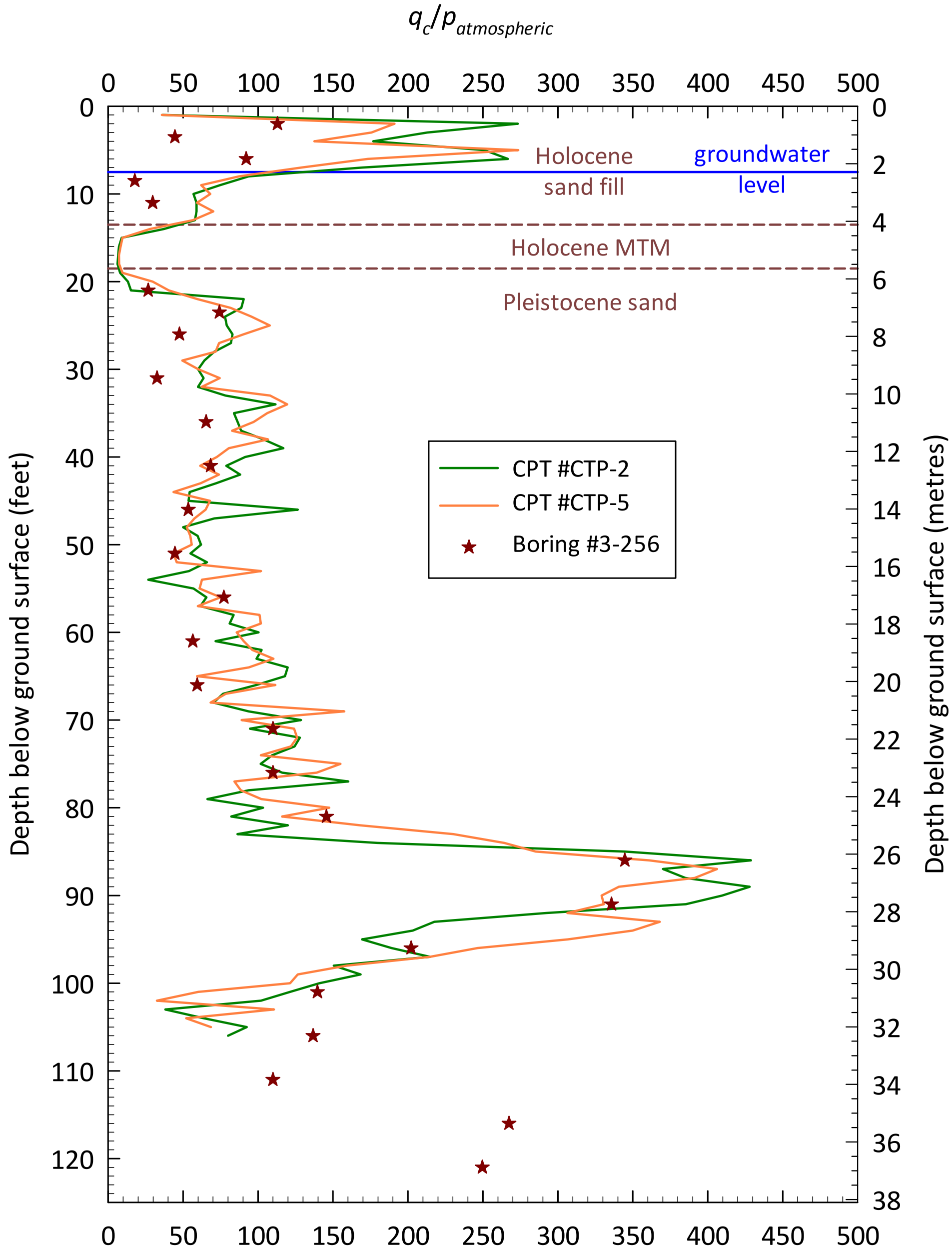

Figure 13. Non-Dimensionalized q_c Values within CTA.

In any event, the conclusions drawn from Figure 13 are that the results from the two CPTs are for all practical purposes the same (which is why they were arithmetically averaged for the plot shown in Figure 11) and that the empirical conversion of N_{60} data to pseudo-q_c data is not bad, especially within the Pleistocene sand stratum that is of greatest interest for both foundation- and liquefaction-assessment purposes.

Figure 14 shows the calculated vertical effective overburden stress, σ'_{vo}, and yield stress, σ'_{vm}[64], both non-dimensionalized to atmospheric pressure, as a function of depth BGS. It is of considerable significance to note that the yield stress of coarse-grain soil is one

[64] A wide variety of names and notations have been used for this parameter over the years. For all practical purposes it is the largest vertical effective stress ever applied to the soil at a given depth.

of several soil properties that have historically been essentially impossible to estimate using either theory or laboratory testing. This fact alone highlights the practical utility and value of in-situ testing in general and CPT/CPTu/sCPTu soundings in particular as geotechnologies that can be used routinely to estimate this important soil property.

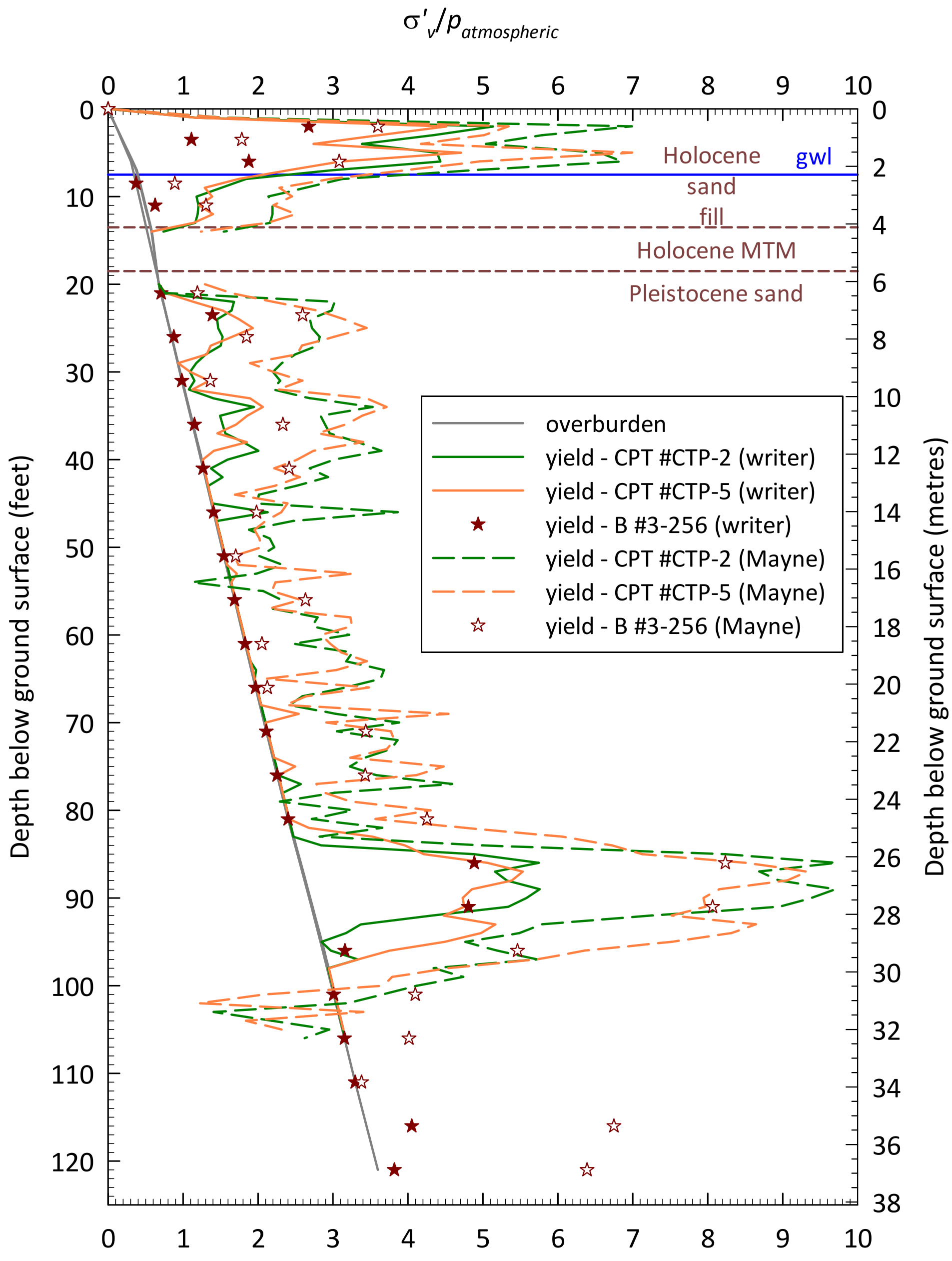

Figure 14. Non-Dimensionalized Overburden and Yield Stresses within CTA.

The reference overburden stress in this figure was produced using the writer's algorithm using soil unit weights calculated by that algorithm. In addition to the three sets of yield-stress results (the two CPTs plus one boring) produced by the writer's analytical algorithm, also shown in this figure are the corresponding three sets of yield-stress estimates obtained using the following relationship proposed in Mayne (2012) and restated in Mayne (2014):

$$\sigma'_{vm} = 0.33 \cdot (q_t - \sigma_{vo})^{0.75} \qquad \textbf{(2)}$$

where all stresses are in kilopascals (kPa) and notation has been changed to be consistent with that used in this paper. Note that the total vertical overburden stress, σ_{vo}, is used in this equation.

Note also that this equation uses the corrected tip resistance, q_t. As noted previously, this parameter could not be calculated for the circa-1988 CPT data used in this paper. Consequently, the uncorrected tip resistance, q_c, was used in Equation 2 instead. However, also as noted previously the difference between the corrected and uncorrected cone tip resistances for terrestrial soundings in coarse-grain soil tends to be relatively small so the error in this case is believed to be small.

Overall, the results from Mayne's empirical correlation consistently indicate somewhat higher values of yield stress and thus greater overconsolidation compared to the outcomes from the writer's algorithm. The writer's results suggest that the Pleistocene sands are, for the most part, normally consolidated under current overburden conditions except for two zones, one near the top of the stratum and other much deeper, Based on recovered SPT samples the deeper zone is marked by a marked increase in coarser soil particles.

On the other hand, Mayne's correlation indicates that the entire thickness of the Pleistocene sands explored are overconsolidated, at least to some extent. Of course in the absence of any other information it is impossible to say which result is closer to reality, if indeed either are. However, it is important to recall that prior to the 1940s placement of fill to create the airport that the overburden stresses would have originated at the top of the Holocene MTM stratum. Therefore, the writer is inclined to favor the results from the writer's algorithm that indicate normally-consolidated conditions for the most part as these would simply reflect the additional overburden stresses caused by fill placement.

Figure 15 shows the coefficient of lateral earth pressure at rest, K_o, as a function of depth BGS. It is again of interest and relevance to note that K_o is another soil property that has historically been essentially impossible to estimate for any type of soil using either theory or laboratory testing thus once again illustrating the practical utility of in-situ testing and results derived from it.

As discussed in greater detail later in this paper, being able to estimate K_o on a site-specific basis is very useful for assessing the geotechnical axial capacity of a deep foundation. Kulhawy (1984, 1991) showed that the lateral earth pressure coefficient, K_h, acting on the shaft of a deep-foundation element after installation, which is linearly related to the unit shaft resistance, r_s, (see Figure 2), can be expressed as the ratio K_h/K_o (where K_o is the pre-installation value in this case) that is a function of the type and installation methodology used for the deep-foundation element. Note that the popular β-method (Fellenius 2012) for calculating the shaft capacity of deep foundations can, in principle, also benefit from improved estimates of K_h as $\beta = K_h \cdot \tan \delta$ where δ is the friction angle between the shaft of the deep-foundation element and adjacent ground.

In any event, the results in Figure 15 are again shown for both the writer's algorithm (which produces this parameter as an explicit calculated outcome) as well as for an extension of Mayne's empirical relationship for yield stress given by Equation 2. Specifically, the fundamental definition of OCR:

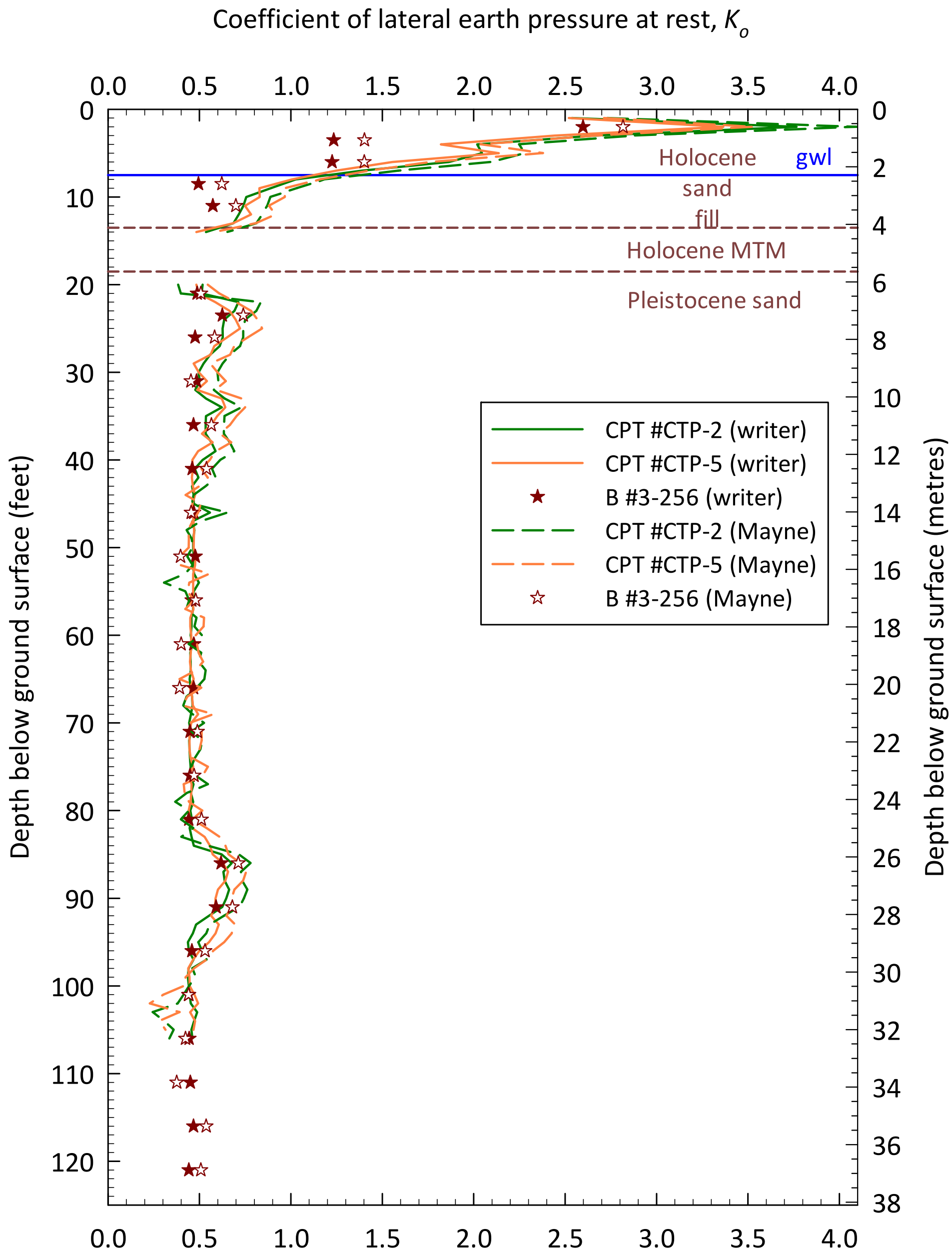

Figure 15. Coefficient of Lateral Earth Pressure at Rest, K_o, within CTA.

$$OCR = \frac{\sigma'_{vm}}{\sigma'_{vo}} \quad \textbf{(3)}$$

can be combined with Equation 2 plus the following empirical relationship in Mayne (2006):

$$K_o = 0.192 \cdot \left(\frac{q_t}{p_{atm}}\right)^{0.22} \left(\frac{\sigma'_{vo}}{p_{atm}}\right)^{-0.31} OCR^{0.27} \quad \textbf{(4)}$$

to produce:

$$K_o = 0.192 \cdot \left(\frac{q_t}{p_{atm}}\right)^{0.22} \left(\frac{\sigma'_{vo}}{p_{atm}}\right)^{-0.31} \left(\frac{0.33 \cdot (q_t - \sigma_{vo})^{0.75}}{\sigma'_{vo}}\right)^{0.27} . \qquad \textbf{(5)}$$

To assist in interpreting the plotted results in Figure 15, it is widely assumed that K_o is a function of a soil's stress state (as defined by its *OCR*) and drained Mohr-Coulomb friction angle, ϕ, according to the following relationship:

$$K_o = (1 - \sin\phi) \cdot OCR^{\sin\phi} . \qquad \textbf{(6)}$$

Until recently, the consensus appeared to be that ϕ should be that for the constant-volume (critical-state) condition, ϕ_{cv}. However, there are indications this is changing.

As shown in the writer's prior publications dealing with JFKIA (Horvath 2002, 2003a, 2004), within the exploration limits defined previously and reflected in Figure 15 ϕ_{cv} is almost constant with depth, with at most a trend of a slight increase with depth. Therefore, any depth-wise variation of K_o at JFKIA would be expected to be exclusively dependent on *OCR* and that is indeed the case observed in Figure 15.

Figure 16 shows relative density, D_r, in percent as a function of depth[65]. This is yet another soil property that has historically been difficult to estimate accurately for coarse-grain soils as correlations based on field *N*-values typically provide only relatively crude estimates at best. Results are shown for both the writer's algorithm (which produces this parameter as an explicit calculated outcome) as well as from the aforementioned *CPeT-IT* software (Version 1.70). This program also calculates relative density as an explicit result using the methodology described in Robertson and Cabal (2012).

The results from both analytical methodologies shown in Figure 16 compare quite well throughout all depth ranges. The use of relative density with deep foundations is discussed subsequently. It is also of use with respect to liquefaction as it provides qualitative insight into potential zones of liquefaction.

A relatively recent development with regard to the consistency of coarse-grain soil is the so-called state parameter, ψ, that is defined as the difference between the natural or existing void ratio, e_n, and the critical-state void ration, e_{cs}, at the same mean effective stress:

$$\psi = e_n - e_{cs}. \qquad \textbf{(7)}$$

Thus ψ becomes increasingly negative as the soil gets denser and its void ratio decreases.

Robertson (2010) expressed the opinion that ψ is a better measure of coarse-grain soil consistency (state) than relative density, with potential correlations and applications related to shear strength and liquefaction among other things. So it is not surprising that there has been considerable research in recent years to relate the state parameter for coarse-grain soils to CPT tip resistance. Empirical correlations for this are included in the aforementioned *CPeT-IT* software and discussed in Robertson and Cabal (2012). A plot of ψ as a function of depth for the two CPT soundings considered in detail in this paper is shown in Figure 17.

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[65] The descriptive terms 'very loose', etc. shown in this figure were adopted from those in Table 2-9 of Kulhawy and Mayne (1990) who cited books by Terzaghi and Peck and Lambe and Whitman as their primary references.

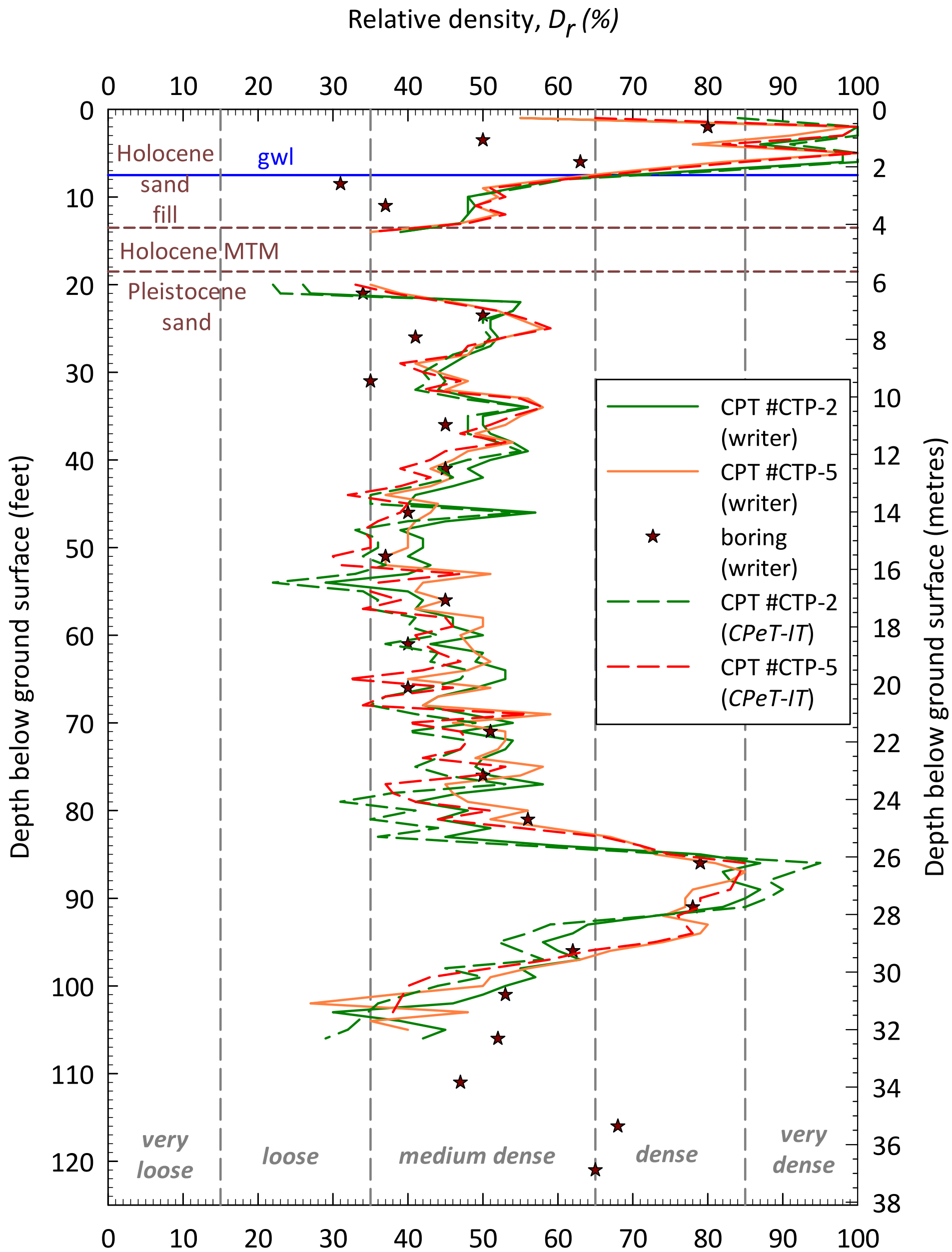

Figure 16. Relative Density, D_r, within CTA.

Other In-Situ Testing Devices

The writer is not aware of any in-situ testing devices other than the aforementioned SPT and CPT (and presumably CPTu in recent years) that have ever have been used at JFKIA. This includes devices such as the dilatometer (DMT) and pressuremeter (PMT) in all their variants that could, in principle, be used and useful in the site conditions present at JFKIA.

The writer does not consider this presumed lack of use of other in-situ devices to be technically deficient for several reasons:

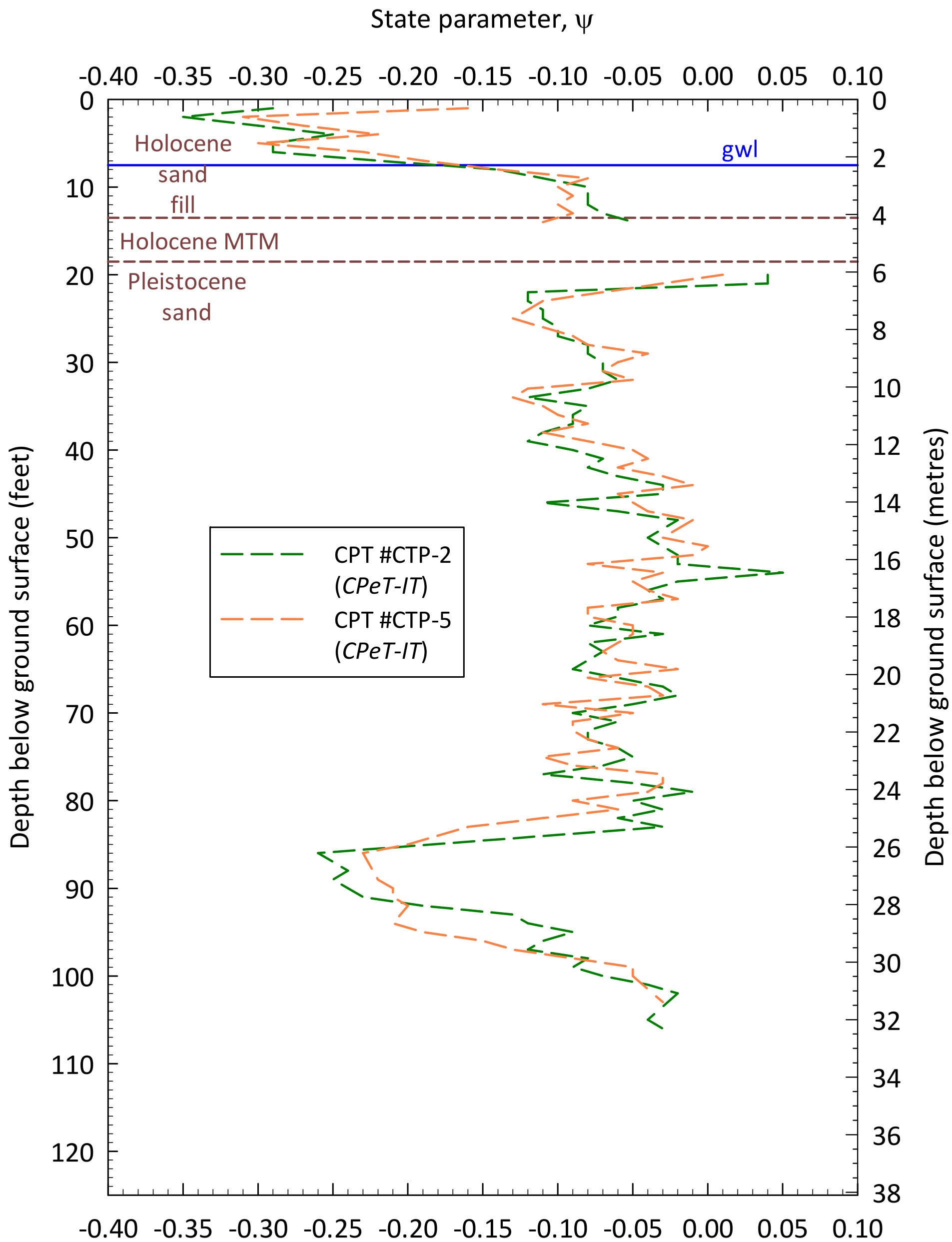

Figure 17. State Parameter, ψ, within CTA.

- As noted previously, CPT/CPTu/sCPTu and SPT (when converted to equivalent q_c values) data can be used to estimate a wide spectrum of soil properties for the soil conditions found at JFKIA. These are also the devices of choice for liquefaction assessments. Thus there is very little of interest in the way of soil properties that is not already possible to estimate with these tools.

- Devices such as the DMT and PMT produce data at discrete depths that are generally relatively far apart compared to the data-sampling frequency of the CPT/CPTu/sCPTu and even SPT. Therefore, devices such as the DMT and PMT cannot compete with the CPT/CPTu/sCPTu and SPT in terms of producing a relatively large volume of data in ground conditions such as at JFKIA where the latter devices perform very well.

That having been said, given the current need to design for seismic loading, which induces relatively significant lateral loads on foundations, there might be some value to performing DMT and/or PMT tests in the Holocene sand fill and/or MTM strata. Both of these devices induce lateral loading within the ground and thus might produce useful results for lateral-load analyses of deep foundations.

For the sake of completeness, it is worth noting that although there may not be a pressing need to use other in-situ testing devices at JFKIA, except, perhaps, for the narrow application re laterally-loaded deep foundations noted above, there is certainly a benefit to keeping up with improvements in CPT technology. In particular, the sCPTu has become much more of a mainstream exploration tool in recent years that should arguably be used for at least some of the CPTu soundings on every project or at least every project at sites without prior sCPTu data. The writer is not aware of any sCPTu testing that has been done at JFKIA.

Soil Mechanics and Foundation Engineering

Introduction

As noted previously, the temporal existence of JFKIA has coincided with the growth and evolution of soil mechanics as well as modern foundation engineering that is based on geomechanics science as opposed to experience-based art. As such, a complete discussion of what has been learned about soil mechanics, and the practice of foundation design and construction based on it, over the last 70-plus years is understandably voluminous and well beyond the scope of this paper. Consequently, the discussion in this paper is limited to those specific issues the writer feels have (or at least could or should have) directly impacted or influenced foundation engineering practice at JFKIA.

This discussion is organized by separate sections that deal with the science of deep-foundation capacity and those that deal with deep-foundation product development. However, all of this is preceded by a discussion of liquefaction which, as will be seen, has evolved in recent decades to be the primary factor governing deep foundations at JFKIA.

Seismic Liquefaction

Introduction

No single geotechnical issue has impacted and altered foundation design at JFKIA more than the evolutionary recognition, understanding, and appreciation of seismic activity in general, and liquefaction in particular, that developed in the final decades of the 20th century and has continued to evolve in the 21st century. A detailed discussion of liquefaction and the common methodologies for assessing its potential occurrence is well beyond the scope of this paper. However, in order to understand the current state-of-practice as it would (or at least should) apply to any liquefaction assessment made at JFKIA some presentation and discussion of the topic is necessary. Both basic concepts as well as more-recent research related to the effects of soil aging will be addressed. After that, a limited discussion of performing liquefaction assessments at JFKIA will be presented.

Liquefaction has always been a consequence of relatively large earthquakes in certain geological settings. However, it is fair to say that the phenomenon came to the forefront in the eyes of geotechnical engineers around the world as a result of major earthquakes in Anchorage, Alaska and Niigata, Japan, both in 1964. The physical damage caused by liquefaction associated with each of these events is still literally textbook material some 50 years later. More significantly, these events led to the circa-1970 development of the cyclic-stress method[66] for performing liquefaction assessments by H. B. Seed[67] and associates (primarily I. M. Idriss[68]) that still forms the backbone of liquefaction assessments both in practice and academic research worldwide.

Initially, the intended use of the methodology was as a predictive tool (i.e. "Will liquefaction occur at this site in the future under an earthquake of such-and-such a magnitude?") and was based on SPT *N*-values. It was subsequently extended to CPT/CPTu/sCPTu q_c values and, more recently, shear-wave velocity, V_s, profiles.

To apply the cyclic-stress method to a given site requires determination of two primary problem variables at one or more depths:

- the cyclic resistance ratio, *CRR*, that is a non-dimensionalized estimate of the soil's maximum shearing resistance against liquefaction and

- the cyclic-stress ratio, *CSR*, that is a non-dimensionalized estimate of the shear stresses induced by the seismic event based on assumptions as to the magnitude and maximum ground-surface acceleration of the design earthquake.

Before proceeding further, it is important to discuss this terminology as there can be substantial confusion when reading the published literature. The heart of the problem is that it has long been common to use the 'CSR' abbreviation for both parameters, distinguishing between them by use of subscripts that are not standardized. For example, the 'true' CSR that represents the driving stress from the seismic event might be labeled $CSR_{EQUATION}$. The other CSR (*CRR* in reality) that represents the resisting stress from the soil is variously described as being the stress necessary to cause liquefaction or maximum liquefaction resistance and labeled CSR_L or CSR_l. Youd and Idriss (2001) called attention to this terminological confusion at the beginning of their paper and recommended the above-defined *CSR*-*CRR* notation that will be used exclusively throughout the remainder of this paper.

In any event, for each piece of in-situ test data (SPT, CPT, V_s) the *CRR* is estimated (only graphically originally) using a curve that delineates the liquefaction and no-liquefaction zones of a Cartesian-plot quadrant. This empirically-derived curve is the central element of the cyclic-stress method. Because it is based entirely on observed liquefaction or

[66] Also referred to as the 'simplified procedure' by H. B. Seed and others in the literature. In recent years, the term 'liquefaction triggering procedure' has appeared in the published literature as an alternative term (e.g. Idriss and Boulanger 2010, Boulanger and Idriss 2014). However, the traditional term 'cyclic-stress method' will be used in this paper.
[67] As will be seen, the distinction between H. B. Seed and his descendent, R. B. Seed, is necessary in the discussion of the cyclic-stress method.
[68] For this reason. the cyclic-stress method is sometimes referred to in the literature as the 'Seed-Idriss method'. For reasons that will become clear later in this paper, this term will be modified slightly to 'HBSeed-Idriss version' (of the cyclic-stress method) and its use will be limited to the original method as developed and updated throughout the 1970s and 1980s up to the point of H. B. Seed's death in 1989.

lack thereof from actual seismic events it has been subject not only to frequent updating over the last 40-plus years based on new seismic events but also reinterpretation of older seismic events by new researchers. In recent years, the concept of 'triggering' liquefaction has emerged in the technical lexicon with associated terms of 'trigger levels' and 'triggering curve', the latter used for the aforementioned curve delineating the liquefaction-no liquefaction zones[69].

The result of estimating *CSR* and *CRR* at as many depths as desired in a given profile of SPT, CPT, or V_s data can be used in various ways:

- The values of *CRR* and *CSR* at each depth can simply be compared in a simple, intuitive maximum-strength versus applied-stress comparison to see which one is larger and thus governs in terms of whether or not liquefaction would be expected at that depth.

- The writer has found plots of calculated shear stresses and maximum shear strength (abscissa) versus depth (ordinate) as illustrated in Kramer (1996) to be a useful academic-instructional tool. These stresses are easily calculated by dividing *CSR* and *CRR* respectively by the vertical effective overburden stress. Wherever the shear strength is greater than the shear stress no liquefaction would be expected and the reverse is true wherever stress exceeds strength.

- Alternatively and nowadays most commonly, it has become popular to calculate and plot a liquefaction safety factor, SF_L, (defined here as = CRR/CSR) as the abscissa versus depth (ordinate). This is particularly useful when the data are depth-wise dense compared to typical SPT profiles such as when CPT soundings are used. Calculating safety factor is also useful because as discussed in many references (e.g. Kramer 1996, TRB 1999) simply concluding that liquefaction is unlikely does not tell the whole story. This is because relatively high pore pressures, which can begin to develop at an early stage in a seismic event (TRB 1999), can develop at safety factors greater than 1 (i.e. liquefaction is not an all-or-nothing phenomenon as the traditional cyclic-stress method plot depicts) and be problematic due to the concomitant non-zero but much-reduced strength of the soil.

While each of these ways of applying the cyclic-stress method has merit in different situations, in order to facilitate comparison between and among analytical results to be presented in this paper the concept of liquefaction safety factor will be used exclusively as a means of portraying the outcomes of the analyses.

A very significant development as time went on is that the cyclic-stress method began to be used in an alternative way, as a forensic tool. Specifically, at sites where liquefaction was known to have occurred it was used in reverse to make a lower-bound estimate of the peak ground-surface acceleration that would have triggered the liquefaction. This is discussed further subsequently.

Another important issue discussed later in this paper is that there have been significant and technically-contentious evolutionary developments in the cyclic-stress method in the decades since initial development of the HBSeed-Idriss version. These evolutionary developments significantly impact application of this method to JFKIA.

[69] While the original version of this plot had a single triggering curve, for some years now the plot typically contains multiple triggering curves for different fines content of the soil. Even more recently, the plot has been modified further so that the multiple triggering curves represent varying levels of probability of liquefaction which marks a significant paradigm shift from earlier versions that were more absolute, i.e. there was either liquefaction or no liquefaction expected.

Soil-Aging Effects

There are two time-related issues involving the cyclic-stress method that are worthy of extended discussion for their relevance to application of this method in general and at JFKIA in particular. Each issue is temporally newer in terms of its recognition and use in practice and thus somewhat less widely known compared to mainstream liquefaction assessment that goes back more than 40 years now. They are:

- paleoseismology/paleoliquefaction and

- temporal effects, usually referred to as soil-aging, on analytical methodologies for calculating various seismic-related phenomena.

The former often provides data for the latter so is discussed first.

Paleoseismology involves geomorphological assessment of visible surface and near-surface physical features for evidence of past seismic events using any number of signature, marker phenomenon associated with seismicity such as liquefaction (paleoliquefaction) and tsunamis. Any such features are then assessed with various scientific tools to back-calculate or estimate parameters such as when the event occurred; what lower-bound magnitude the event might have had; and what lower-bound maximum ground-surface acceleration might have occurred that would otherwise be unknown based on human history alone.

The net outcome of this forensic exercise is to increase the temporal database of significant seismic events in a given geographical region beyond that which can be constructed based on recorded human history alone as well as to provide additional data for the aforementioned plots with triggering curve(s) to estimate the potential for future liquefaction. The benefit of this forensic exercise in areas such as the Eastern U.S. in general, and NYC metropolitan area in particular, where the seismic record is relatively sparse in geological time is potentially significant.

The single biggest detriment to utilizing paleoseismology as a geotechnical tool in any given area is human development which, with few exceptions, alters the ground surface so that any evidence of past seismic events is either physically destroyed or buried beyond reasonable access. Nevertheless, paleoseismology has been applied successfully in some areas, including the Northeastern U.S., although to date the latter has been primarily within New England (Tuttle 2006)[70] in areas most affected by the regionally-significant Cape Ann earthquake of 1755 that is estimated to have been in the magnitude, M, low-6 range which, from an energy perspective, is approximately 10 times larger than any recent event in the NYC metropolitan area.

The writer is not aware of any paleoseismological studies that have been performed anywhere within the NYC metropolitan area in general or JFKIA in particular. It is quite possible that the Jamaica Bay area may have once contained paleoliquefaction features from the aforementioned 1884 event and possibly other, earlier events. Earthquakes with M in the low-5 range, such as those that are believed to have impacted the NYC metropolitan area at least several times in the relatively recent geological past, are believed to be at the low-end of the range that can cause liquefaction under the right conditions. However, the land areas within and surrounding Jamaica Bay have been drastically altered by a combination of human activity and continued sea-level rise so that paleoseismological features are unlikely to be found in the future.

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[70] The complete proceedings of both the 2006 and 2012 USGS CEUS workshops noted in this reference can be accessed at: earthquake.usgs.gov/hazards/about/workshops/CEUS-WORKSHP/ (accessed 6 November 2014).

With regard to the primary issue of the effect of soil aging on the cyclic-stress method, a very important distinction to make in the discussion of temporal effects as they relate to liquefaction is geological age versus behavioral age. The geological age of a soil deposit is always a fixed quantity and relates to the time before present when the soil was either deposited by a natural process or placed as a result of some human activity. As long as a soil deposit remains where it is its geological age never changes.

On the other hand, the behavioral age relates to the most-recent time before present when the soil fabric or structure was completely disturbed or remolded by some event such that it destroyed all stress memory and any particle cementation of the soil, in a sense resetting its 'biological clock'. This would obviously happen when a soil was initially deposited naturally or as the result of some human activity. But it would also happen again naturally in the context of earthquakes due to liquefaction or as the result of some human activity such as blasting, deep dynamic compaction, or vibroflotation after initial deposition or placement as would occur as a result of ground modification/improvement. Thus a geologically 'old' soil can have a behaviorally 'young' age. Furthermore, unlike geological age the behavioral age can be reset an unlimited number of times.

The reason for making this age distinction will be seen subsequently as the behavioral difference of what are often referred to as young/uncemented soils versus old and/or cemented soils in terms of liquefaction potential is now recognized. Thus when it comes to liquefaction assessment in addition to estimating the geological age of a soil deposit it becomes desirable to try to estimate its behavioral age as well.

As noted previously, the cyclic-stress method can be used in two distinctly different ways:

- In its traditional role as a <u>predictive</u> tool as discussed previously in some detail.

- In its more-recent role as a <u>forensic</u> tool which is a logical extension of the method. For example, if a site-specific paleoseismological investigation reveals paleoliquefaction features that would unequivocally indicate past liquefaction then based on the current profile of in-situ test data a lower-bound estimate of ground-surface acceleration that caused the liquefaction can be attempted.

Olson et al. (2001) discuss the broader issues with using the cyclic-stress method for both types of applications although they focus most of their paper on the latter forensic aspect. More-recent work by Gassman et al. (2004) and Leon at al. (2006) focuses more narrowly on the effects of aging on soil behavior.

The writer's interpretation and understanding of the key points made in these papers are as follows:

- The original and still-primary intent of the cyclic-stress method is as a predictive tool for assessing future liquefaction potential at a specific site for a specific earthquake magnitude and assumed peak ground-surface acceleration using field data obtained in the present. However, because the triggering curve(s) that is/are the cornerstone of the methodology is/are entirely empirical in nature the overall method has always relied on data from sites where liquefaction is believed to have either occurred or not as a result of an earthquake that occurred in the current timeframe. This has been necessary not only to acquire the necessary subsurface data (SPT N-values, etc.) but also to have accurate estimates of earthquake magnitude (specifically moment magnitude, M_W) and peak ground-surface acceleration based on actual measurements. This means that the acquired subsurface data were always obtained <u>after</u> the earthquake that caused or did not cause the observed liquefaction. As discussed in some detail by Olson et al. (2001), there are several distinctly different physical phenomena that can actually work against each other in terms of their effects on soil structure after liquefaction completely reworks the fabric or structure of a soil, i.e. resets the behavioral age of the soil. As a result, it is possible, in principle at least, that post-event soil properties at a site (*N*-values in particular) can be either greater or less than those that existed pre-event. According to Olson et al. (2001), H. B. Seed and his associates were aware of this[71] so attempted (in some unspecified fashion) to correct the post-event *N*-values to values they felt would have existed pre-event. This is consistent with their wanting to develop a predictive tool for use on sites where presumably liquefaction has never occurred and thus altered the soil structure. Apparently, later revisions and versions of the cyclic-stress method dispensed with this attempt to backdate *N*-values. As a result, the *N*-values (or CPT q_c values or V_s profiles) that appear in current versions of the method are simply values obtained in the present with no attempt to backdate them pre-event. In the writer's opinion, this is a troubling aspect of this method in that whether or not liquefaction occurs at a site depends solely on conditions existing at the time of the earthquake. Yet to use a predictive method that is based on conditions measured after liquefaction has occurred with no attempt to relate these conditions to those that existed pre-liquefaction seems illogical (an opinion apparently shared by others). Nonetheless, this is the current state-of-practice.

- Olson et al. (2001) devoted most of their paper to developing the logic for five different 'scenarios' as they called them for backdating/calculating pre-liquefaction *N*-values for a site. However, their reason for doing so did not relate to the original predictive mode for the cyclic-stress method but the second, later forensic use to estimate a lower-bound value for the maximum ground-surface acceleration that caused the liquefaction. It is both enlightening and disconcerting that they found that even using just one *N*-value at one site the range in calculated accelerations varied by ±50% about the mean (average) of the highest and lowest values depending on which of the five *N*-value back-dating scenarios they used.

- A relatively narrow, yet still-important, issue raised by Gassman et al. (2004) and Leon et al. (2006) is that the empirical relationships developed for the cyclic-stress method have, from the beginning, been biased toward sites where liquefaction occurred in relatively young soils of Holocene age. As discussed extensively by Olson et al. (2001), Gassman et al. (2004), and Leon et al, (2006), it has long been recognized that all soils age in the sense that their mechanical (stress-strain) properties change over time due to a complex and incompletely-understood combination of mechanical (due to physical particle rearrangement) and chemical (due to the development of inter-particle cementation as the result of the precipitation of dissolved solids in the groundwater) phenomena. The consequence of this aging is that Gassman et al. (2004) and Leon et al. (2006) show that Pleistocene and older soil deposits appear to have a significantly increased resistance to liquefaction due to the combined effects of aging. The relevance of this is that the soils at JFKIA with the greatest potential for liquefaction are those within the Pleistocene stratum directly beneath the Holocene MTM (see Figure 11).

[71] Current researchers are clearly aware of and sensitive to this issue as well. For example, Boulanger and Idriss (2014), who focused on the CPT as the preferred in-situ testing tool, discuss the issue of before and after results from CPT soundings at the same site.

In the writer's opinion there are two issues that do not appear to have been addressed explicitly in any of these published discussions in the literature concerning the overall phenomenon of soil aging that are relevant to JFKIA. One is the effect of long-term changes in groundwater chemistry, specifically, from freshwater to brackish water with varying amounts of oceanic salinity. The Pleistocene soils of the Upper Glacial Aquifer that underlie the Holocene MTM stratum and form the bearing stratum for all deep foundations at JFKIA were presumably deposited in a freshwater fluvial environment that was well above sea level at the time of deposition. During the subsequent rise in sea level to the present, freshwater would have remained in this aquifer as long as the piezometric level remained above sea level. However, because this aquifer has been exploited by humans for potable-water supply the piezometric level has dropped in places so that the porewater has become brackish due to saltwater intrusion from the adjacent Atlantic Ocean. This is the reverse of the phenomenon that produces quick-clays (wherein freshwater replaces saltwater over time). How this change in porewater chemistry may affect soil cementation in the long term is unknown to the writer.

The other issue is the effect of relatively significant changes in vertical effective stress due to overburden addition or removal. The importance is that assessments of liquefaction potential in the present do not relate to liquefaction potential in the past when the effective-stress regime was significantly different. In the case of JFKIA, there has been a significant increase in vertical effective stresses within the natural soils underlying the Holocene sand-fill stratum due to the substantial filling that occurred to create the airport property. Of interest here is whether liquefaction of the Pleistocene sands may have occurred in the past when vertical effective stresses were almost 2 kips/ft^2 (100 kPa) smaller in magnitude. Such a happenstance would influence the estimate of the behavioral age of these soils.

On a separate issue, it is of interest to note that the sCPTu mentioned earlier in this paper has promise as a tool to assist geotechnical engineers with assessing the behavioral age of a soil deposit. It has been found (Robertson 2014) that the shear-wave velocity, V_s, profile of young/uncemented soils can be estimated quite accurately using empirical correlations (e.g. Robertson and Cabal (2012) and as implemented in the *CPeT-IT* software) applied to basic CPTu data. However, when the estimated V_s profile deviates significantly from a measured profile obtained using the sCPTu then this suggests a soil that is behaviorally old and/or cemented as such soils exhibit greater small-strain stiffness that would be apparent from shear-wave measurements.

Applications at JFKIA

It is of interest to explore the issue of performing site-specific liquefaction assessments at JFKIA, at least to a limited extent using the subsurface information considered in this paper. The writer cautions that the results of this exercise presented in this paper are by no means intended to be a definitive assessment of liquefaction potential at JFKIA and no conclusions along these lines should be drawn from what is presented herein. What follows in this paper is intended primarily to illustrate the current state of uncertainty that is inherent in liquefaction assessments due to the many levels of subjectivity involved in the making a liquefaction assessment.

To begin with, for some years now the New York City Building Code[72] has included seismic-related provisions for certain categories of new construction. The current base

[72] publicecodes.cyberregs.com/st/ny/ci-nyc/b200v08/index.htm?bu=YC-P-2008-000006. Accessed 6 November 2014.

version of the Code was issued in 2008 with updates issued subsequently. Section BC 1813 (titled "Liquefaction Analysis") of Chapter 18 (titled "Soils and Foundations") of the current version of the Code mandates that liquefaction be addressed for every building site but allows two very different levels of assessment to be performed at the discretion of the engineer-of-record for the project:

- A simplistic analysis that is applied to all saturated "noncohesive" soils down to a depth of 50 feet (15.2 m) BGS. The details of this analysis will be presented and illustrated subsequently.

- Per the Code: "*A site-specific analysis performed by an engineer with specific expertise in the evaluation of liquefaction*.". The details of this analysis are not specified although the aforementioned cyclic-stress method is clearly allowable if not intended by virtue of additional verbiage used in the Code. This analysis is subject to review and approval by the Commissioner of the NYC Building Department.

The default simplistic analysis allowed by the Code will be addressed first. It is reflected in Figure 1813.1[73] in the Code that is reproduced in its essential elements in this paper as Figure 18. Note that the 50-foot (15.2 m) depth limit used in this figure is an explicit component of the Code procedure. This likely derives from the fact that the original HBSeed-Idriss version of the cyclic-stress method considered depths beyond 15 metres (49.2 ft) as "unverified" for the method (Youd and Idriss 2001).

To begin with, the criteria for dividing between zones of "probable" versus "unlikely" liquefaction vary with a dimensionless parameter called Structural Occupancy Category (*SOC*) that ranges from I to IV in order of increasing relative importance of the building. The types of structures in each Category are defined explicitly in Table 1604.5[74] of Section 1604 ("General Design Requirements") of Chapter 16 ("Structural Design").

Note that Category I structures are de-facto exempt from liquefaction assessment which is why there is no line shown for that Category in Figure 18. At JFKIA, most buildings would fall into either Category III (e.g. terminals with relative large human-occupancy levels) or Category IV (e.g. public-safety facilities such as fire stations plus the aircraft-control tower).

In the writer's opinion, the item of greatest geotechnical interest and concern with regard to this methodology is that the Code uses field N-values (N_{field}) as the in-situ assessment parameter, a fact that the writer finds both surprising (in this day and age of enlightenment re SPT N-values) and troubling. Troubling because at any given depth N_{field} can vary by a factor of two or more based solely on the efficiency of the hammer-drive system. Thus at a given depth in a given soil a completely different conclusion re liquefaction potential could be inferred simply by virtue of the hammer-drive system employed.

[73] publicecodes.cyberregs.com/st/ny/ci-nyc/b200v08/st_ny_ci-nyc_b200v08_18_sec013.htm?bu=YC-P-2008-000006. Accessed 6 November 2014.
[74] publicecodes.cyberregs.com/st/ny/ci-nyc/b200v08/st_ny_ci-nyc_b200v08_16_sec004.htm?bu=YC-P-2008-000006. Accessed 6 November 2014.

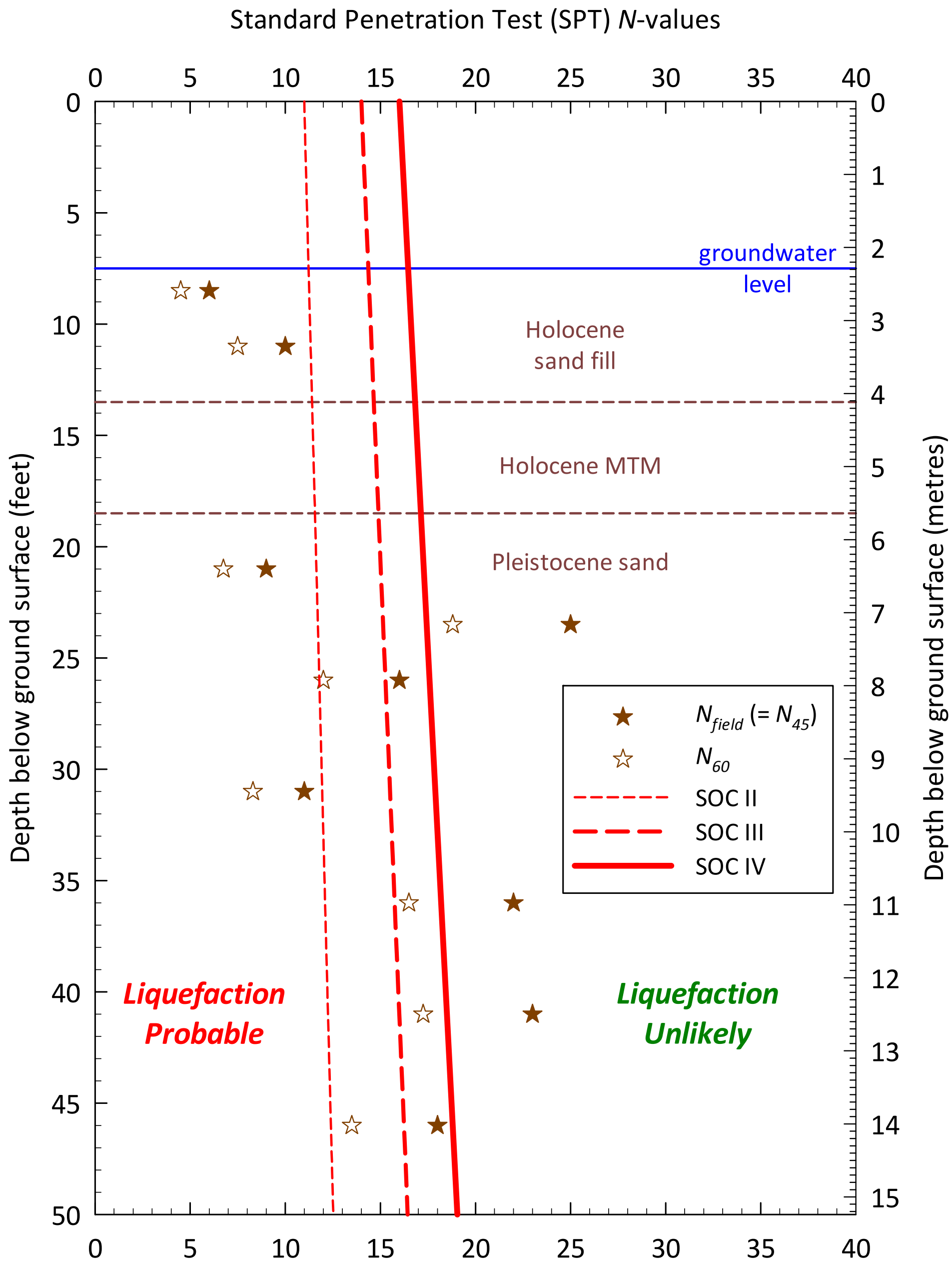

Figure 18. NYC Building Code Liquefaction Assessment Applied to Boring No. 3-256.

Even worse, in the writer's opinion, is the fact that the way the Code is structured it actually encourages the use of an inefficient hammer-drive system because the less efficient the system the larger the N_{field} and the less likely that liquefaction probability will be inferred. This is completely contrary to the way in which any simplified design or analysis procedure should be crafted, i.e. any methodology crafted with simplicity-of-use as its primary basis should always err on the conservative, 'safe' side, not vice versa as in this case.

This point is amply illustrated in Figure 18 using the data from Boring No. 3-256 within the CTA. Both the field *N*-values, which are assumed to be equal to N_{45} values based on the hammer-drive system used in this boring, and hypothetical N_{60} values, which could have been achieved in principle even in 1988 when this boring was drilled simply by using a different hammer-drive system, are shown, Note that there are several N_{field}-N_{60} data pairs that straddle both the *SOC* III and IV lines that would cover virtually all buildings at JFKIA.

In any event, the conclusion one would draw from using this default Code methodology is that liquefaction is probable both within the saturated portion of the Holocene sand-fill stratum as well as within significant portions of the Pleistocene sand stratum within which all deep foundations at JFKIA derive support.

Next considered are more-advanced analyses based on the cyclic-stress method. As will be seen, this exercise is actually fraught with significant uncertainty, perhaps more so than at any time in the past 40-plus years since the method was first promulgated, and requires some discussion of the backstory[75].

The current situation of significant analytical uncertainty may seem surprising as one might think that time would have reduced, not increased, uncertainty as a result of progressive technical refinement over time of an analytical methodology. Stated another way, as time goes on one might expect a technical subject to become more, not less, accurately known as more knowledge is produced. However, as is not uncommon in the process of emergence followed by evolutionary growth of a technology, when it first appears there is usually one relatively simple way to do things that has been put forth by one person or at most a relatively small group of collaborators who are all of like mind, on the same proverbial 'wavelength' if you will, so that getting 'the' correct deterministic[76] answer is straightforward and unambiguous. In time, that answer may actually turn out to be wrong in the absolute sense but that is a separate issue.

With regard to the original (circa-1970) HBSeed-Idriss version of the cyclic-stress method, it arguably reached its pinnacle of unqualified acceptance circa 1985 when a workshop of experts in the field was convened under the auspices of the National Research Council (NRC) and produced a consensus document (NRC 1985). However, as often happens when technology is involved, as time goes on more and more people from outside of the original group that first developed the technology get involved and soon there are divergent opinions as to how to achieve 'the' answer. As a result, there comes a point where that which was straightforward originally no longer is so. This happened with the cyclic-stress method, especially after H. B. Seed's sudden and unexpected death in 1989. As Youd (2011) stated, "*Chaos was beginning to develop*" as the 1990s evolved.

In an effort to address this "chaos", there was a major overhaul of the cyclic-stress method that had broad agreement on the outcome contents among leading experts in the field. This revision process was conducted over a period of several years in the late 1990s beginning with another workshop, this one in 1996 and hosted by the National Center for Earthquake Engineering Research (NCEER). Key elements of the outcomes from this effort can be found in TRB (1999) and Youd and Idriss (2001).

For identification purposes in this paper, what emerged from this late-1990s effort will be referred to as the Youd-Idriss version of the cyclic-stress method. It is relevant to note that this version contained some notable evolutionary changes to the SPT *N*-value triggering curves compared to the final HBSeed-Idriss version of circa 1985 although the

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[75] The writer cannot state strongly enough that no opinion or taking-of-sides are expressed or implied in the discussion that follows as the writer 'has no horse in this race' other that being an end-user of the technology. The following discussion is presented solely to inform and educate the reader and to illustrate the current level of uncertainty and subjectivity faced by design professionals wishing to perform a site-specific liquefaction assessment using the cyclic-stress method.

[76] In this context 'deterministic' means single-valued as opposed to a 'probabilistic' answer that is a range of answers, each with a different probability of occurrence. As will be seen, the issue of deterministic versus probabilistic answers has emerged with liquefaction assessments in recent years.

basic methodology was unchanged. There were also some significant additions in terms of recognizing the increasing use of CPT soundings as well as V_s profiles for alternative versions of triggering curves.

As an aside, as noted previously I. M. Idriss was the primary collaborator and colleague of H. B. Seed during early developmental work of the cyclic-stress method in the 1970s and 1980s. As such, he can be viewed as the one living continuous link to the past and the 'keeper of the flame' of the original HBSeed-Idriss version of the cyclic-stress method. Consequently, Idriss and later research collaborators (T. L. Youd and, more recently, R. W. Boulanger) can be viewed collectively as comprising one school-of-thought concerning progressive improvements to the cyclic-stress method as it evolved from the HBSeed-Idriss method of the 1970s and 1980s to the Youd-Idriss version of the late 1990s to the Boulanger-Idriss versions of the present. Simply stated, this school-of-thought has been to tweak H. B Seed's original simplified method here and there from time to time without making any radical or wholesale revisions to the methodology (one can assume that none was felt necessary), in essence periodically simply making 'a good thing better'.

In any event, it was not far into the 21st century before this methodology consensus that had coalesced around the Youd-Idriss version in the late 1990s began to unravel as a result of significant and significantly different reinterpretations of the same case-history data set by a group led by H. B Seed's son, R. B. Seed (with K. O. Cetin and R. E. S. Moss apparently key collaborators). This divergence-of-opinion all seems to have come to a head as a consequence of the publication in 2008 of a monograph by Idriss and Boulanger (2008) that produced what will be called the Boulanger-Idriss/2008 version of the cyclic-stress method. Publication of this document led to a 'paper war', the opening salvo of which was a research report (Seed 2010) and follow-up lecture tour and email campaign by R. B. Seed that apparently and primarily took strong exception to the technical content of the Idriss-Boulanger monograph. Specifically, a RBSeed-Cetin-Moss version of the basic clean-sand triggering curve based on SPT *N*-values that delineated the liquefaction-no liquefaction zones of the traditional cyclic-stress method plot was significantly more conservative than anything shown heretofore in the HBSeed-Idriss or Youd-Idriss versions. This requires some elaboration to understand how this evolved.

The primary difference between the legacy versions of H. B. Seed's original work (HBSeed-Idriss, Youd-Idriss, Boulanger-Idriss/2008) and the RBSeed-Cetin-Moss version is illustrated succinctly in a presentation by Youd (2011). Historically, a single basic (clean-sand) triggering curve was used to separate the liquefaction-no liquefaction zones of the plot where *CRR* is the ordinate and *N*-value is the abscissa. R. W. Seed et alia chose a radically new and different approach by presenting a family of clean-sand triggering curves, each of which presented a probability of liquefaction, P_L, ranging from 5% to 95%. This is one way to address the fact that there had always been several case-history data points that fell on the 'wrong' side of the traditional plot with a single triggering curve. Stated another way, the data had always suggested that there is more of a transition zone where liquefaction might, but not always, occur as opposed to the abrupt all-or-nothing implication of a single triggering curve. Thus the approach taken by R. W. Seed et alia can be viewed objectively as simply a mathematically more-elegant way to depict what in reality has always been there, i.e. there is a transitional zone of uncertainty of liquefaction.

However, when using this novel form of the triggering curves proposed by R. W. Seed et alia problems arise from the fact that in practice a deterministic, as opposed to probabilistic, approach is typically used which means that a design professional needs to choose a single triggering curve to use from the several provided to calculate, say, SF_L. Toward that end, R. W. Seed et alia apparently recommended that the P_L = 15% curve be used for this purpose. Note that this is a relatively conservative choice, most likely chosen to err on the side of over-predicting liquefaction potential within the transitional 'gray' zone that has always existed.

As shown by Idriss and Boulanger (2010), Youd (2011), and Boulanger and Idriss (2014), this P_L = 15% curve is substantially lower than the Boulanger-Idriss/2008 version that appeared in Idriss and Boulanger (2008) which was virtually the same as the Youd-Idriss version produced by the 1990s consensus (Youd et al. 2001). The net result is that for a given set of conditions the RWSeed-Cetin-Moss version produces liquefaction safety factors that are almost always significantly (of the order of 30% on average) lower than those produced using the Youd-Idriss or Boulanger-Idriss/2008 relationships. As noted by O'Rourke (2011), the cost implications of this (in terms of ground-modification or other treatment methods necessary to mitigate potential liquefaction hazards) are substantial, hence the widespread concern among all liquefaction stakeholders in practice.

There are other points of disagreement between the two current assessments of what is essentially the same database. For example, R. W. Seed et alia see greater uncertainty in certain aspects of calculating the *CSR* when determining the shear-stress-reduction factor, r_d. However, the aforementioned difference in the trigger curves and the concomitant effect on the *CRR* and SF_L appear to be the single most significant and contentious point of disagreement.

In addition to strictly technical issues, there were apparently other, peripheral issues. For example, R. B. Seed apparently also took exception to the fact that the Idriss-Boulanger monograph was published under the auspices of the Earthquake Engineering Research Institute (EERI). The implication is, apparently, that the Idriss-Boulanger monograph in general, and in particular the Boulanger-Idriss/2008 version of the cyclic-stress method and *N*-value triggering curve that it presented, somehow had the imprimatur and backing of EERI as a de-facto manual-of-practice. However, the O'Rourke et al. (2010) report that was commissioned and prepared to deal with this professional controversy found no such linkage. In fact, there was apparently language in the Idriss-Boulanger monograph that made this clear although, apparently, not clear enough for R. W. Seed:

> *"...any opinions, findings, conclusions, or recommendations expressed herein are the authors' and do not necessarily reflect the views of FEMA or EERI."*

In any event, the only issues of interest in this paper are the technical ones and not any of the apparent personalized acrimony that was also alluded to in a subsequent presentation by O'Rourke (2011). The fact that peripheral issues exist is mentioned solely to illustrate the thorny, contentious landscape concerning liquefaction assessments that currently exists that those involved in making such assessments, whether as a practitioner or academic researcher, have to navigate.

On the other side, in 2010 Idriss and Boulanger produced a follow-up (to their 2008 monograph) research report (Idriss and Boulanger 2010) that was essentially a formal rebuttal to the R. W. Seed research report (Seed 2010). In particular, Idriss and Boulanger addressed in detail several particular case histories that seemed to 'drive' the location of the family of probabilistic trigger curves in the RWSeed-Cetin-Moss version of the cyclic-stress-method method away from the more-traditional Boulanger-Idriss/2008 triggering curve. Idriss and Boulanger (2010) suggested that if certain corrections were made with regard to interpretation of these case histories then the RWSeed-Cetin-Ross triggering curve for P_L = 15% would be much more in consonance with the Boulanger-Idriss/2008 version of the curve. The writer is not aware of any subsequent publication(s) by R. W. Seed or associates that address this issue.

In any event, Idriss and Boulanger recently (2014) produced yet another update (Boulanger-Idriss/2014 version) of the cyclic-stress method to take advantage of substantial new case-history data that resulted from significant seismic events in New Zealand in 2010 and 2011 and Japan in 2011. This update was contained in a research report (Boulanger and Idriss 2014). This report does not cite any recent (i.e. post-2010) publications by R. W. Seed or his associates.

However, the most significant aspect of this 2014 report is that for the first time it focuses on the CPT, not SPT, as being the in-situ test method of choice for liquefaction assessments. Most of the report deals with CPT correlations for triggering curves with relatively little content devoted to the traditional SPT methodology. This may signal a low-awaited shift away from the SPT as the primary in-situ tool to use for liquefaction assessments.

As a final comment on this issue of controversy surrounding cyclic-stress methodologies, to the best of the writer's knowledge the recommended (O'Rourke at al. 2010, O'Rourke 2011) new workshop that would be convened to resolve the necessary technical issues between the Boulanger-Idriss/2008 (but by now 2014) and RBSeed-Cetin-Ross versions of the cyclic-stress method has not occurred or even been scheduled. As a result, the burden is on the design professional to choose an analytical method when performing a site-specific liquefaction assessment.

Returning now to site-specific issues, a detailed comparison between, and evaluation of, the various versions of the cyclic-stress method for conditions at JFKIA and/or how triggering curves based on the various in-situ test methods (SPT, CPT, V_s) compare for a given set of subsurface conditions is well beyond the scope of this paper. However, a limited suite of analyses will be presented to provide at least some basic insight into issues that practitioners should be aware of and consider in practice.

The starting point of any liquefaction assessment using any version of the cyclic-stress method is to define the design seismic event in terms of magnitude, M[77], and maximum ground-surface acceleration, a_m. Based on the discussion presented earlier in this paper, M = 5.25 and a_m = 0.24g were selected for use for all analyses performed for this paper.

It is worth noting that this is a relatively high acceleration for an event of this magnitude (as noted previously, bedrock acceleration for an earthquake of this magnitude in the NYC metropolitan area would be expected to be in the range of 0,14g to 0.15g) and reflects the NEHRP Class D site-amplification factor applied by codes. Therefore, in practice it would be worthwhile for a design professional to consider performing a more-rigorous analysis of the transmission of bedrock motions up through the soil column to produce an alternative estimate of maximum surface acceleration and/or site-specific values of CSR to use in liquefaction analyses.

It is also significant to note that this magnitude is at the low end of the range for which the cyclic-stress method has been developed. More importantly, at this low end of the range the data used to create the various empirical relationships that are essential components of the cyclic-stress method are sparse, widely scattered, and have been subject to widely-varying, ever-changing interpretations and recommendations over the years. This is especially true of the magnitude scaling factor, MSF, that has a direct influence on CRR. These issues are explored in greater detail later in this paper.

Three sets of liquefaction analyses were performed for this paper using the computer program *CLiq*[78] (pronounced 'slick') that is commercially available from the same

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[77] Youd and Idriss (2001) recommended that only moment magnitude, M_W, be used for liquefaction assessments using the cyclic-stress method. Recent publications such as Idriss and Boulanger (2010) and Boulanger and Idriss (2014) followed this advice in the analyses they performed for these studies.

[78] Version 1.7.6.34

vendor as *CPeT-IT*. The *CLiq* software allows the user to choose from quite a few (10) versions of the cyclic-stress method: three for SPT data, five for CPT data, and two for V_s data. For some selections there are additional parameter choices that can be made as well so that the potential number of analytical versions that can be used is even greater than 10. So all in all the burden is on the user to select a method or methods to use on a project.

There are some general comments about the various plotted results from *CLiq* presented subsequently in figures:

- The various versions of the cyclic-stress method are labeled in each figure using the nomenclature in *CLiq* which, in some cases, may differ from that used by the writer for the same version. This was done intentionally to allow readers familiar with or acquiring (in the future) *CLiq* to correlate the plotted results with that software.

- All plots show SF_L as a function of depth BGS. No results are shown below a depth of 80 feet (24.4 m) because of increasing uncertainty with depth for the cyclic-stress method in general and r_d factor that affects *CSR* in particular. There is also a noticeable increase in the relative density and coarseness of the soils that begins around this depth (see Figures 11, 16, and 17) that makes liquefaction less likely at greater depths.

- The *CLiq* software arbitrarily places an upper-bound cap of SF_L = 2 on all calculations so the various plots presented subsequently in this paper are scaled accordingly. This causes some of the data points and curves to 'stack up' on top of each other in some of the plots.

- Both the SF_L = 1.0 line (solid red) and SF_L = 1.5 line (dashed red) are shown. As discussed previously, significant excess pore pressures and concomitant significant reductions in shear strength can develop even if liquefaction does not occur. Based on information shown in Kramer (1996) and TRB (1999) it appears that excess pore pressures build rapidly once SF_L drops below approximately 1.5 so the range of SF_L between 1.0 and 1.5 can be interpreted as being a transitional zone of caution as significant strength reductions of the soil can be expected.

- The writer elected to use the Boulanger-Idriss/2014[79] version of the cyclic-stress method for CPT data as the primary analytical tool as this version appears to incorporate the most-recent data from significant liquefaction events in Japan and New Zealand where the CPTu was the in-situ testing tool of choice. This version also identifies the CPTu (and sCPTu of course) as the in-situ tool of choice for liquefaction assessments in the future so it is expected that future development of the cyclic-stress method will focus on the CPT format.

The first set of analyses considers the present conditions without any modification for soil-aging effects. To illustrate the range of results that reflects the diverse choices available in the current state of practice, analyses were performed using SPT data from boring No. 3-256; CPT data from both sounding Nos. CTP-2 and CTP-5; and V_s data (not measured directly but calculated from the CPT soundings using *CPeT-IT*).

To begin, Figure 19 shows the results using SPT data. In general, the results from all versions are in reasonable agreement in that none suggests any significant, extensive zones of liquefaction although in some cases significant zones of strength loss (1.0 < SF_L < 1.5) are

[79] The *CLiq* software refers to this version as "Boulanger & Idriss (2014)".

indicated. Overall, the calculated results are considerably less dire than the conclusions drawn from using the simplistic method in the NYC Building Code (Figure 18).

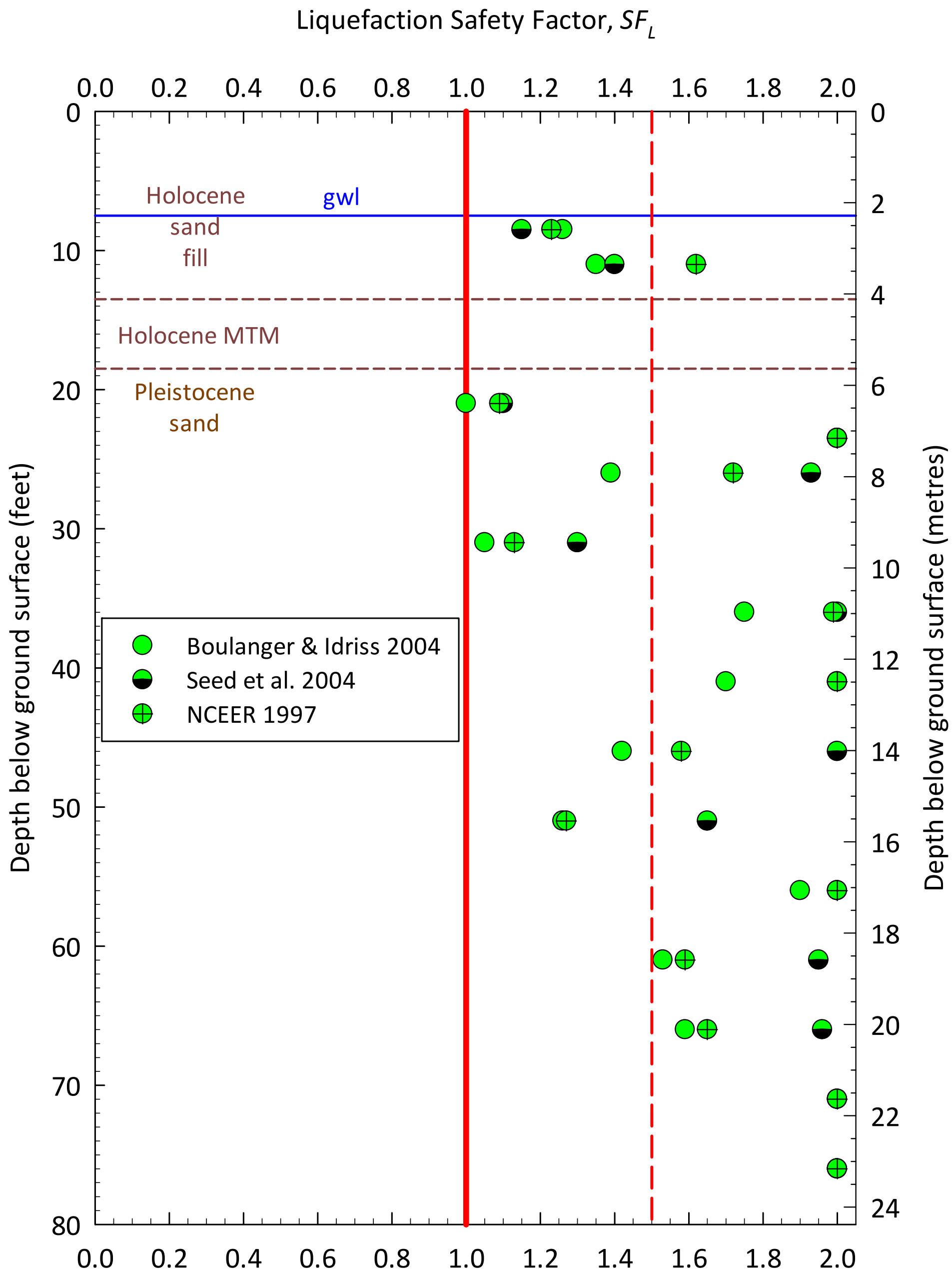

Figure 19. Liquefaction Assessment:
Present Conditions/No Aging/SPT Data - Overall Comparison of Versions.

Next are two plots showing the results using CPT data. Figure 20 compares the results between the two soundings using the baseline Boulanger-Idriss/2014 version of the cyclic-stress method. The desired result from this is to show that the two soundings produce essentially identical results. Consequently, all subsequent CPT-based plots will only use comparisons based on sounding No. CTP-2.

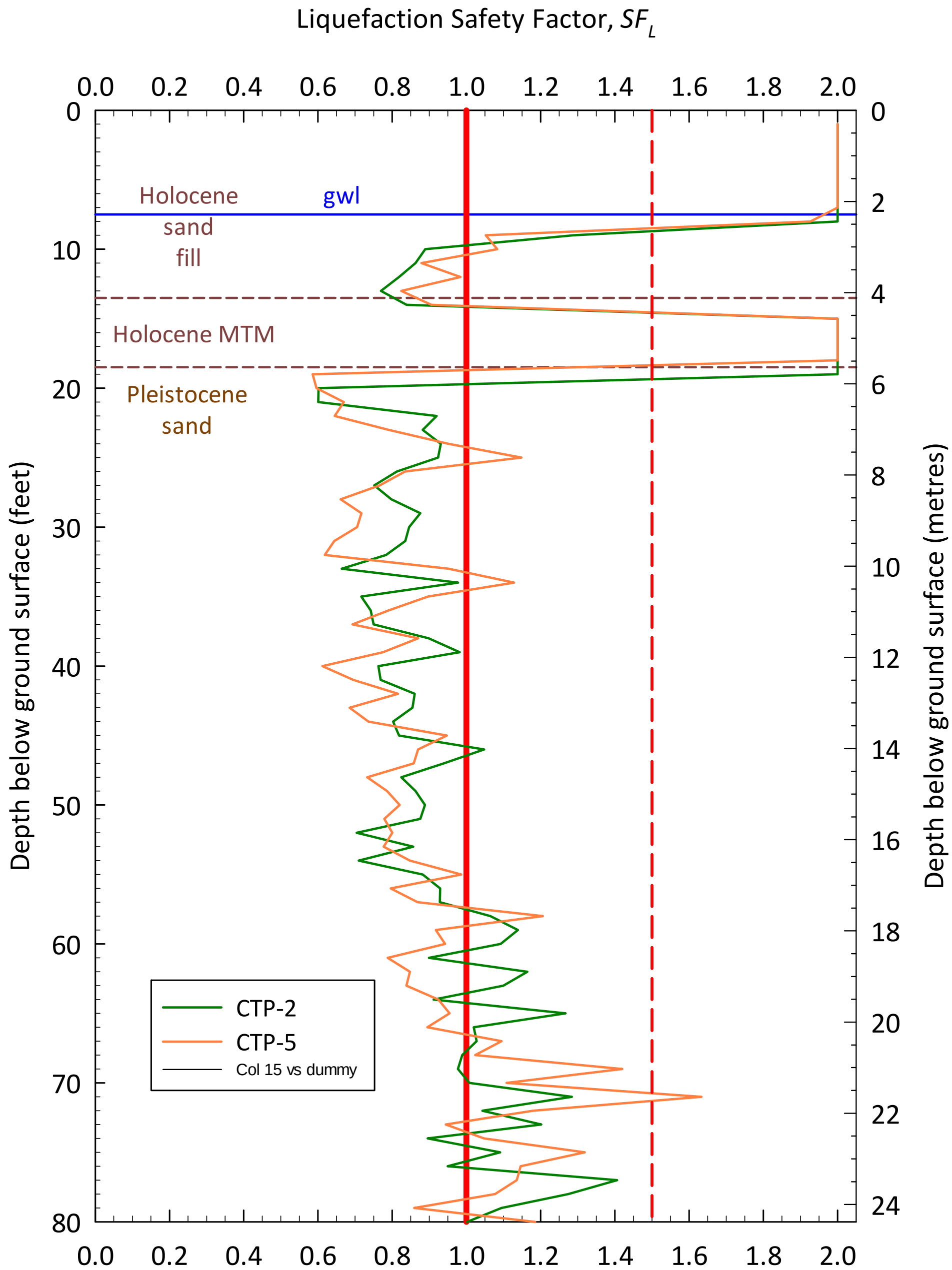

Figure 20. Liquefaction Assessment:
Present Conditions/No Aging/CPT Data - Boulanger-Idriss/2014 Version.

Figure 21 compares the results from the five CPT-based versions of the cyclic-stress method that are available for use in *CLiq*. Parameter variations, where applicable, within a given method were not investigated. Typically only the default parameter settings for a given method were used.

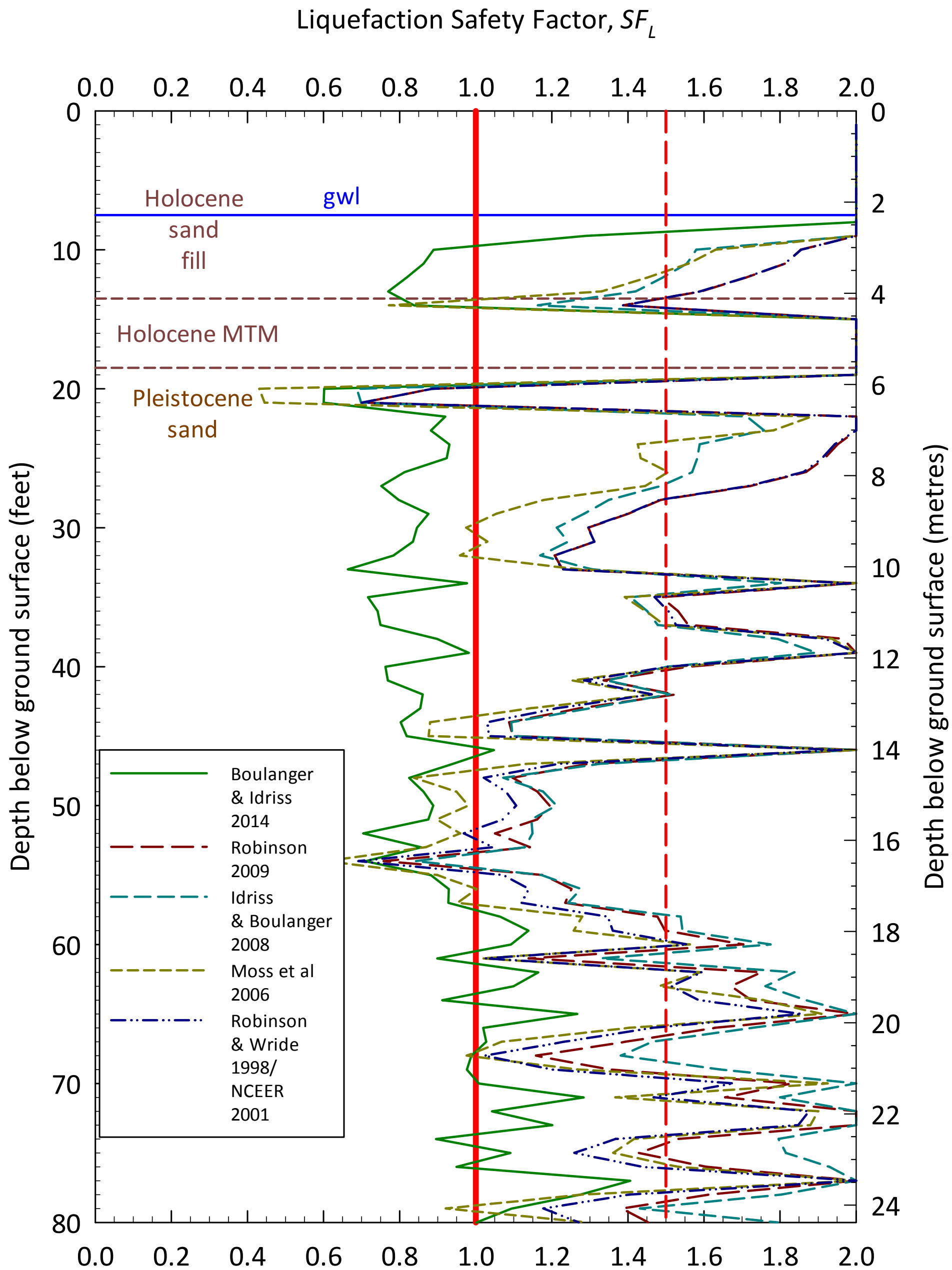

Figure 21. Liquefaction Assessment:
Present Conditions/No Aging/CPT No. CTP-2 Data - Overall Comparison of Versions.

As can be seen, there is a marked difference in results between the most-recent version (Boulanger-Idriss/2014) and the other four versions that cover the 15-plus years before that which are all very similar in their results. The Boulanger-Idriss/2014 version indicates a substantial, continuous zone of probable liquefaction within the upper portion of the Pleistocene sand stratum that the other four versions do not. Interestingly, the Boulanger-Idriss/2014 results are broadly in agreement with the simple, default SPT-based analysis allowed by NYC code (Figure 18) whereas the results from the other four CPT-based versions are broadly in agreement with the SPT-based versions of the cyclic-stress method (Figure 19).

Given that the Boulanger & Idriss/2014 version of the cyclic-stress method both represents the latest thinking on the subject and produces markedly more-conservative results compared to CPT-based versions (and SPT-based as well) in use for some years now, it is of some interest to parse these results to see, at least on a preliminary basis, what the source(s) of this variation may be in terms of the key variables that affect the calculation of SF_L. This is done using several figures, each of which shows the results from the same five versions of the cyclic-stress method shown in Figure 21.

Before doing this, it is important to note that recent CPT-based versions of the cyclic-stress method have all been based on CPTu data as the u_2 pore-pressure data are used to calculate various intermediate parameters. Because the writer prepared this paper using only CPT data, the assumption of hydrostatic porewater pressures within all saturated coarse-grain soil strata was made so that pseudo-u_2 values could be calculated and input accordingly. This was assumed to be reasonable but in reality CPTu soundings in coarse-grain soil typically measure some perturbations about the theoretical hydrostatic-pressure line as a result of varying soil density and fines content. The extent to which pore pressures measured using a CPTu or sCPTu at JFKIA would produce results different from those presented herein is obviously unknown to the writer.

The first parameters examined are *CRR* (Figure 22) and *CSR* (Figure 23) as these calculated parameters are used explicitly to calculate the desired end result, SF_L. Before commenting on the results shown in these figures, it is important to describe the way in which *CLiq* determines and portrays these two parameters.

As shown by Youd and Idriss (2001), SF_L for sites with a planar, horizontal (level) ground surface can be calculated using the following equation (notation has been changed to be consistent with that used in this paper):

$$SF_L = \left(CRR_{7.5} / CSR\right) \cdot MSF \cdot K_\sigma \qquad \textbf{(8)}$$

Although it makes no difference algebraically, from a conceptual or theoretical perspective of visualizing the meaning of the variables shown in Equation 8 Youd and Idriss (2001) noted that *MSF* has been and can be viewed as either modifying or adjusting:

- $CRR_{7.5}$ (the maximum shearing resistance/shear strength of the soil for a M = 7.5 earthquake that long ago was chosen as the baseline event for which the triggering curve(s) have been developed and plotted) or

- *CSR*.

However, they note that the traditional perspective (that the writer prefers as well) used by the original HBSeed-Idriss version of the cyclic-stress method is to use *MSF* to scale $CRR_{7.5}$ to what is sometimes referred to as a field value of *CRR*, CRR_{field} (= $CRR_{7.5} \cdot MSF$).

There is usually less ambiguity with regard to the K_σ parameter which, by definition, empirically normalizes the soil's shearing resistance for overburden-stress effects. Consequently, this parameter should always be visualized as modifying $CRR_{7.5}$ or CRR_{field}.

That having been said, the developer of *CLiq* chose to apply both *MSF* and K_σ to *CSR* in the portrayed tabulation and plotting of results within the program. This means that *CRR* = $CRR_{7.5}$ only is shown and plotted in *CLiq* so this is what is shown in Figure 22.

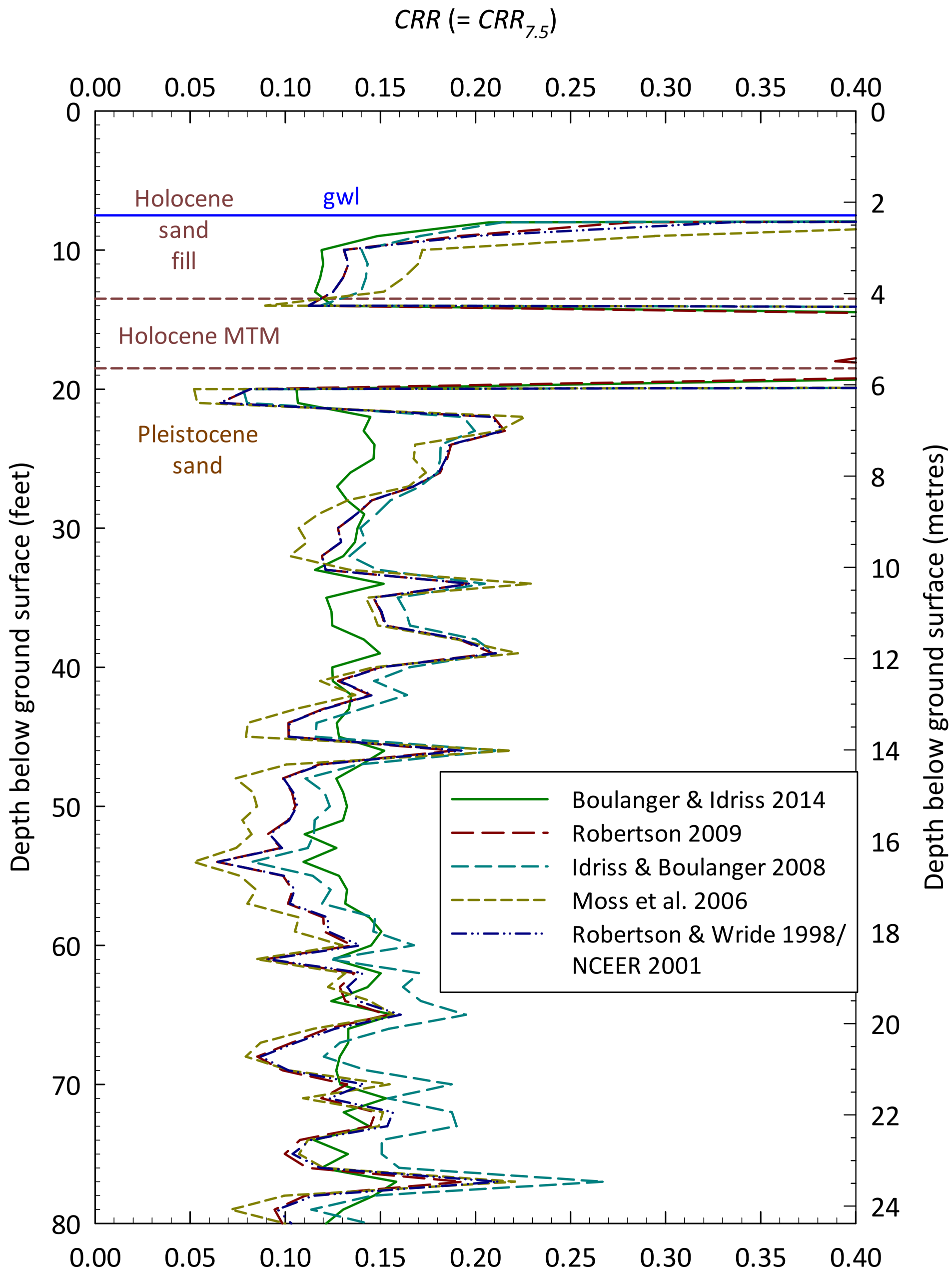

Figure 22. Liquefaction Assessment:
Present Conditions/No Aging/CPT No. CTP-2 Data - *CRR* (= $CRR_{7.5}$) Comparison.

From the perspective of efficiently comparing results, this lumping of variables actually makes eminent sense because when one sees trigger curves published for all three types of in-situ data (SPT, CPT, V_s) they are invariably the basic $CRR_{7.5}$ curves. Consequently, calculating and only showing $CRR_{7.5}$ values facilitates comparison with published trigger curves from different versions of the cyclic-stress method. Nevertheless, it is important to understand how these variables are intended to be lumped together from a conceptual or theoretical perspective.

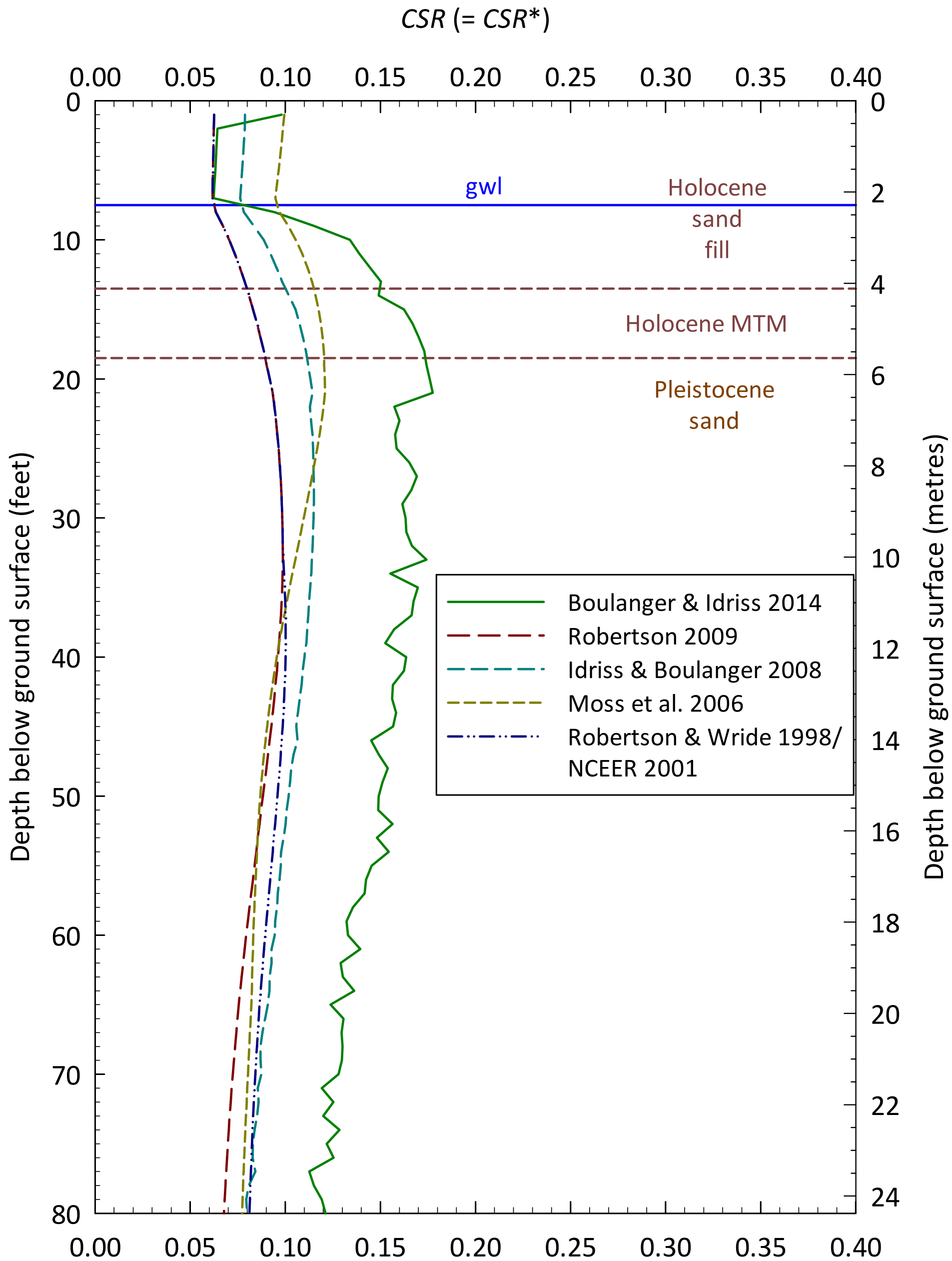

Figure 23. Liquefaction Assessment:
Present Conditions/No Aging/CPT No. CTP-2 Data - *CSR* (= *CSR) Comparison.**

One final comment with regard to Figure 22 is that *CLiq* caps the calculated value of $CRR_{7.5}$ at 4 so some of the plotted results, primarily within the vadose zone and Holocene MTM stratum, plot off-scale. The writer felt it was more important to use a scale in this figure that focused on the lower end of the range which is where all the saturated coarse-grain soils lie as this is what is of greatest interest here.

With regard to Figure 23 and *CSR*, the *CLiq* program tabulates both the basic value as well as final value, *CSR**, that the software developer calls the fully-adjusted cyclic-stress ratio, defined as follows:

$$CSR^* = \frac{CSR}{MSF \cdot K_\sigma} \tag{9}$$

It is CSR^* that is shown in Figure 23 and used in *CLiq* to calculate SF_L (= $CRR_{7.5}/CSR^*$).

As can be seen in these figures, the range in $CRR_{7.5}$ values between and among the five CPT-based versions of the cyclic-stress method than span 15-plus years of R&D is relatively small indicating that basic trigger curves have not changed all that drastically over the years. However, the newest version (Boulanger-Idriss/2014) does 'stray from the herd' somewhat with noticeably lower values of soil resistance, especially within the shallower portion of the Pleistocene sand stratum. This is, of course, precisely the zone where this method suggests much greater probability of liquefaction compared to the other four versions as shown in Figure 21.

There is relatively much greater variation in CSR^* with the results from the Boulanger-Idriss/2014 version substantially different and larger in magnitude than the four earlier versions that all cluster together. Because SF_L decreases as CSR^* increases this variable contributes as well to the lower SF_L values for this version.

It is of interest to pursue the source of this variation in CSR^* as there are only three variables that can differ between and among the five versions of the cyclic-stress methods shown. With reference to Equation 9 these are:

- *MSF*,
- K_σ, and
- r_d (which is embedded in the calculation of the base value of *CSR* and reflects how shear stresses vary with depth relative to a value at the surface, i.e. zero depth).

It is well known that each of these parameters, *MSF* and r_d in particular, have been the subject of ongoing research and discussion for at least the period of time reflected in the five versions of the cyclic-stress method considered in this paper. This is especially true with regard to the values of *MSF* to use with relatively low-magnitude earthquakes such as considered here.

Figure 24 shows the comparison of *MSF* values. Note that the two versions by Robertson are the same so the curves plot on top of each other. There is a relatively large range of values that reflects the variation in opinions concerning this parameter that has occurred over the years. It is of interest to note that the Boulanger-Idriss/2014 version breaks with tradition in that *MSF* is not assumed to be constant as a function of depth as has been assumed historically.

In any event, the variation in *MSF* is certainly one source of the divergence in results for CSR^* shown in Figure 23. Note per Equation 9 that the smaller the value of *MSF* the larger the value of CSR^* so this appears to at least partially explain why the Boulanger-Idriss/2014 version has markedly larger values of CSR^* in Figure 23.

Figure 25 shows the comparison of K_σ values. Note that two versions of the cyclic-stress method assume K_σ = 1 which effectively means this variable does not influence the calculated results. Of the remaining three versions, the absolute and relative variation is modest and appears to have only a modest influence on the variation in CSR^* reflected in Figure 23.

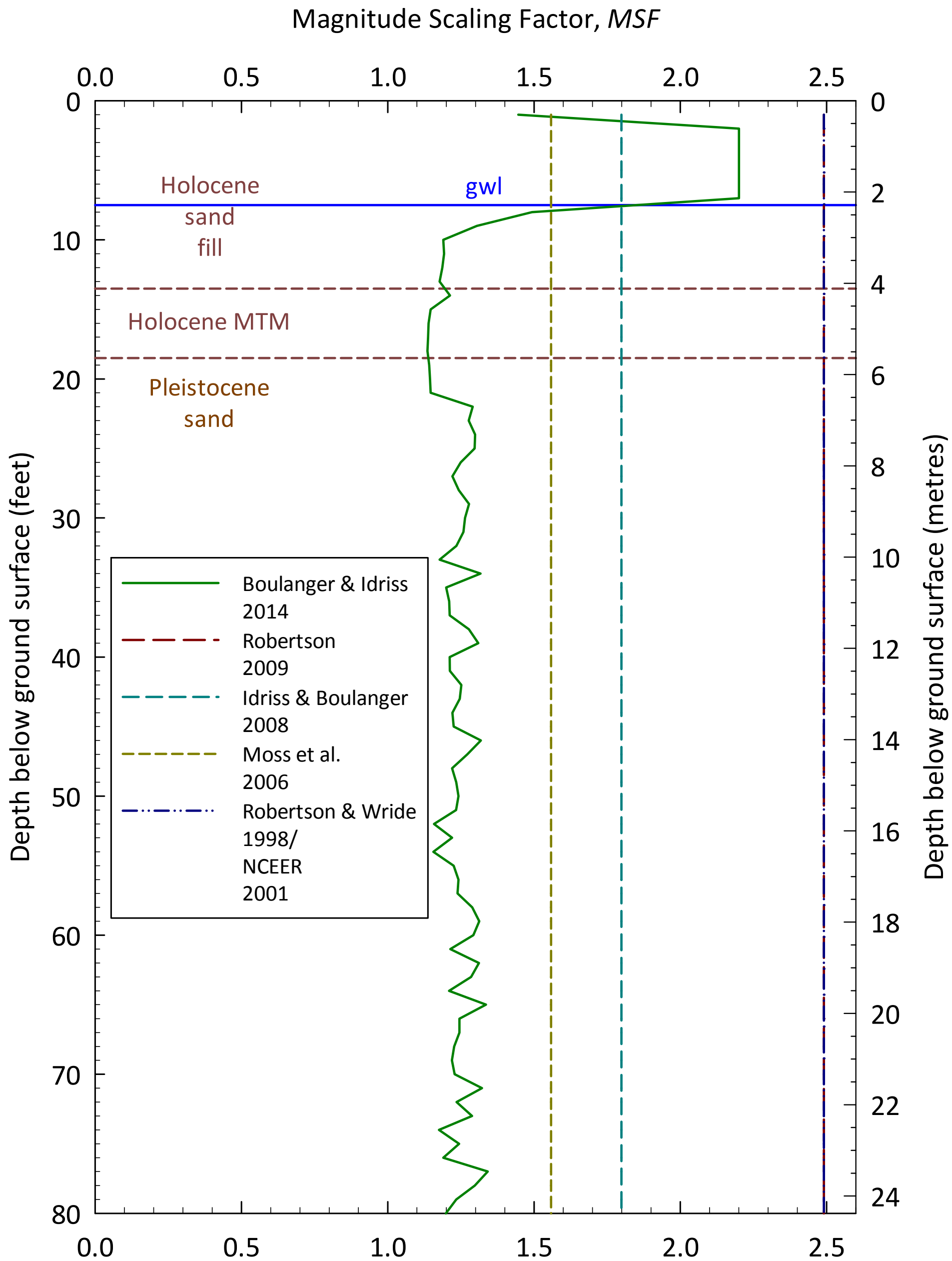

Figure 24. Liquefaction Assessment:
Present Conditions/No Aging/CPT No. CTP-2 Data - *MSF* Comparison

Figure 26 shows the variation in r_d values. Note that the two versions from Idriss and Boulanger use the same relationship and the two versions from Robertson use the same relationship which is why only three independent curves show on the plot. Not surprisingly, these three curves show substantial relative variation with depth. From the earliest days of the cyclic-stress method there has always been a considerable range in proposed values of this parameter that shows a tendency for the width of the range to widen with depth which is one of the reasons for the aforementioned increasing uncertainty in calculated SF_L with increasing depth.

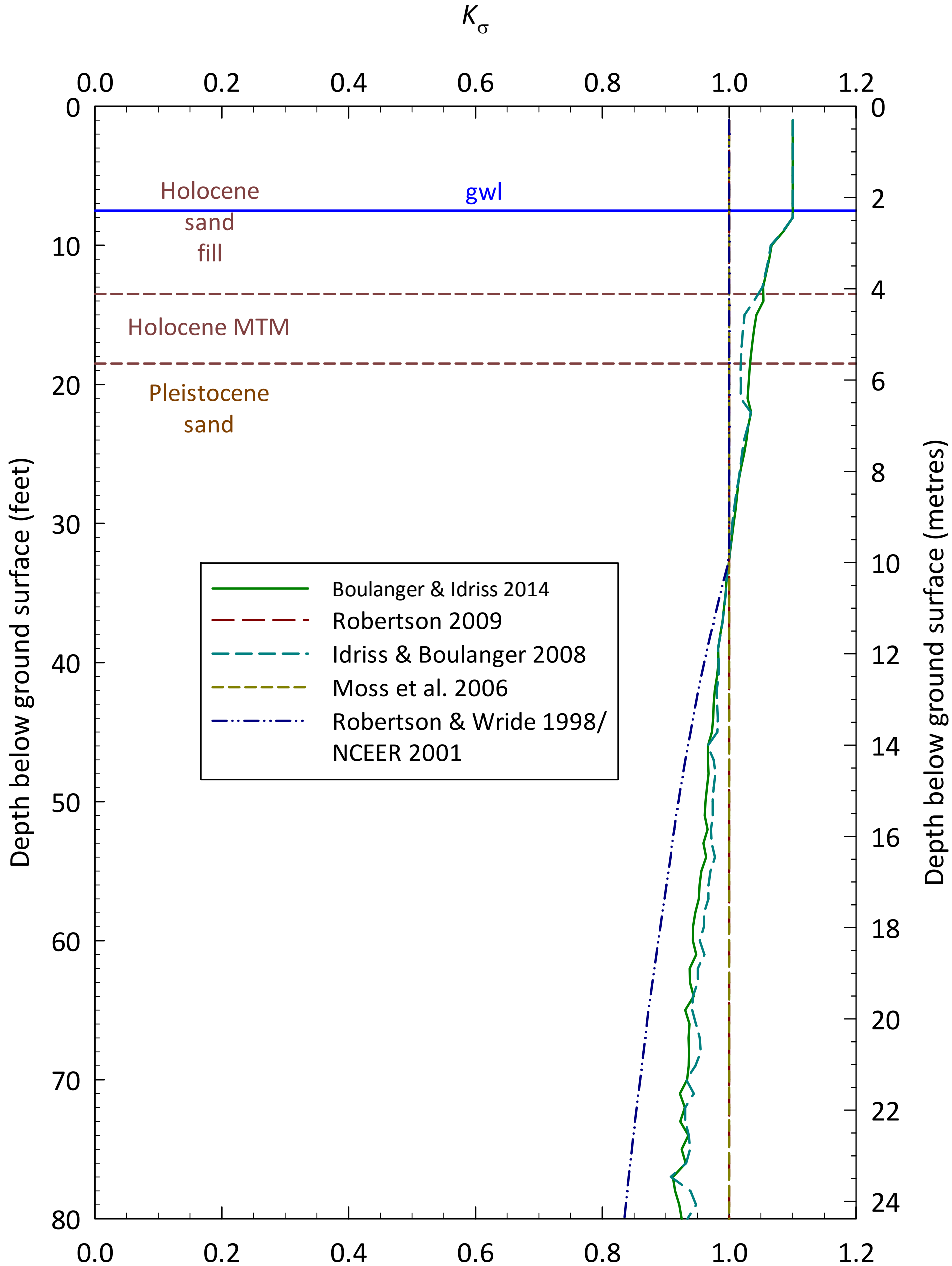

Figure 25. Liquefaction Assessment:
Present Conditions/No Aging/CPT No. CTP-2 Data - K_σ Comparison

In summary and conclusion with regard to the calculated values of SF_L using CPT data shown in Figure 21, it appears that the primary reason the Boulanger-Idriss/2014 version of the cyclic-stress method produces values significantly lower than the other four versions considered in this paper is due to significantly larger values of CSR^*. Essentially, much larger driving stresses are forecast by this version compared to versions proposed over the preceding 15-plus years. Furthermore, the primary cause of this increase in driving stresses appears to be the relatively low values of magnitude scaling factor, MSF, even for a relatively modest assumed earthquake of M = 5.25.

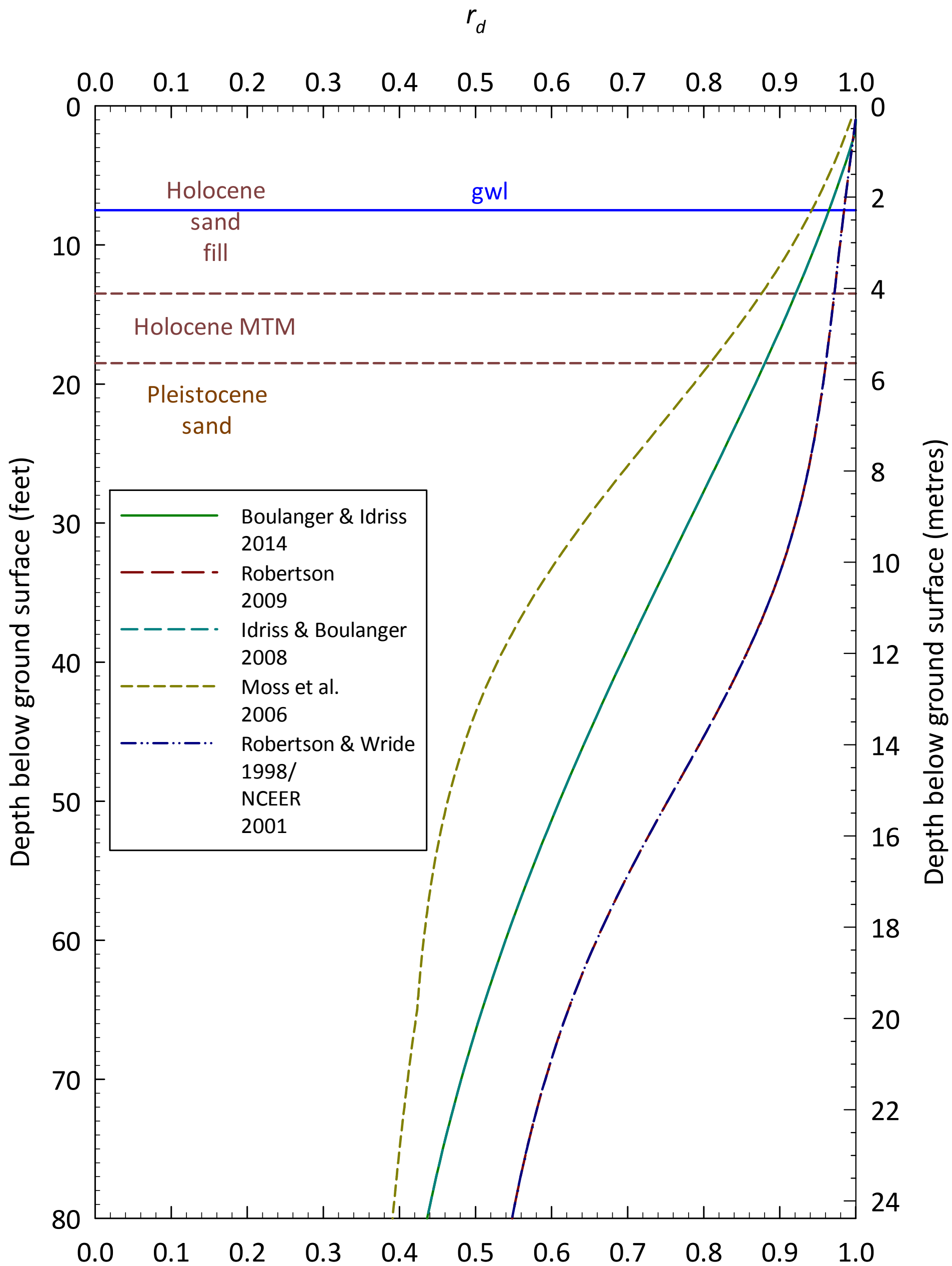

Figure 26. Liquefaction Assessment:
Present Conditions/No Aging/CPT No. CTP-2 Data - r_d Comparison

Figure 27 shows the final assessment made for existing conditions using two versions of the cyclic-stress method that are based on shear-wave velocity, V_s, data. As noted previously, these data were estimated using an empirical algorithm in *CPeT-IT* that is based on young/uncemented soil behavior as opposed to using actual V_s data obtained in the field which are not known to be available for JFKIA. As noted previously, this empirical correlation has proven to correlate very well with actual sCPTu data for young/uncemented soils (Robertson 2014). Of course nowadays in practice it would always be desirable to obtain site-specific V_s profiles using a sCPTu as this device has entered the mainstream of practical in-situ exploration tools.

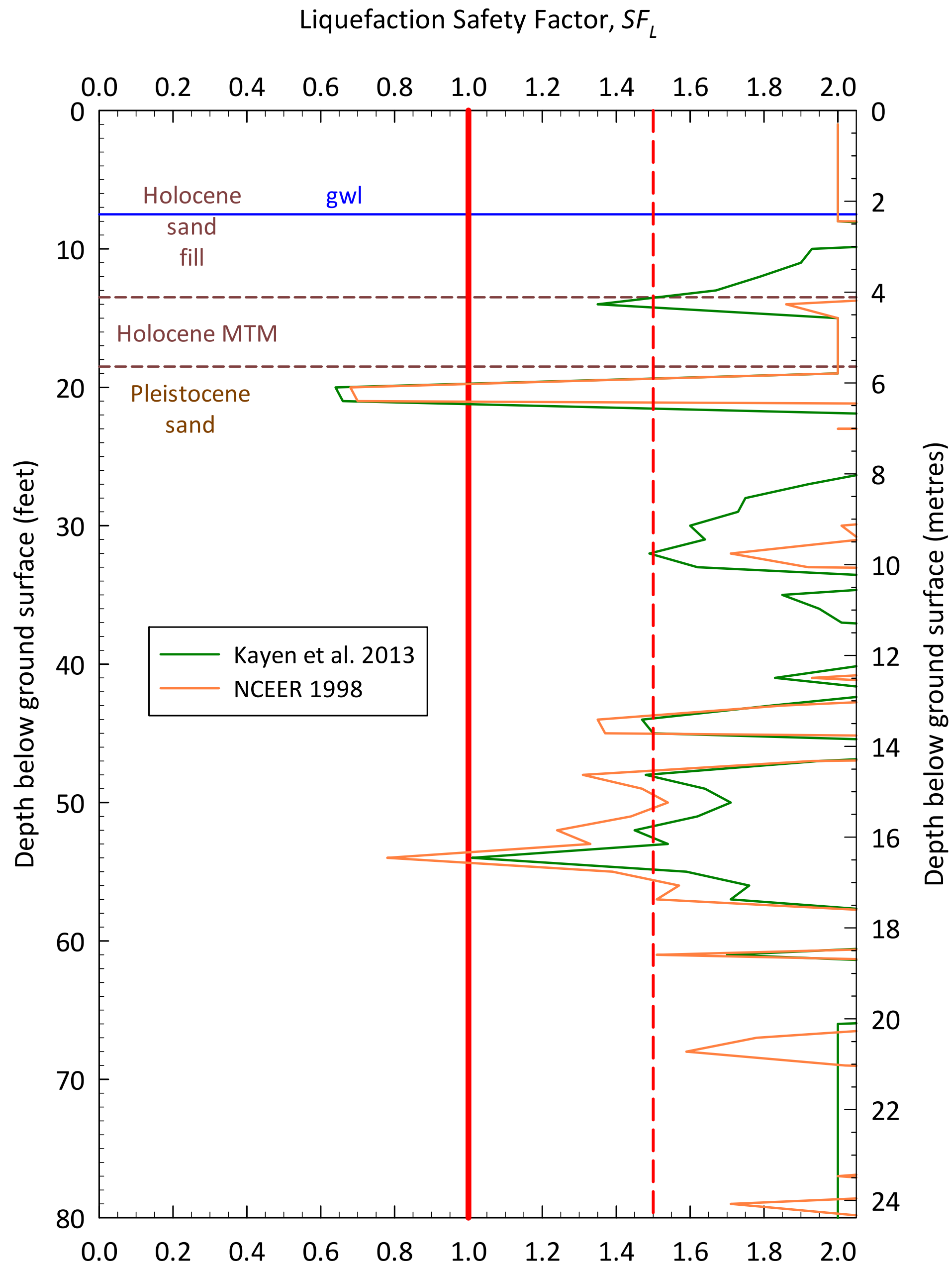

Figure 27. Liquefaction Assessment: Present Conditions/No Aging/ V_s Data Inferred from CPT No. CTP-2 - Overall Comparison of Versions.

To close out the subject of liquefaction assessment under present site conditions, Figure 28 is a 'plot of plots' that shows the results of all 10 versions of the cyclic-stress method shown previously in Figures 19, 21, and 27. The individual versions are intentionally not labeled in this figure as the desire was to detect broad trends of results simply from a visual density of data. There appears to be a broad indication of probability of liquefaction at the very top of the Pleistocene sand stratum and also just below a depth of about 50 feet (16 m). Note that these depths correspond well with zones of lower relative density and positive-value state parameter shown in Figures 16 and 17 respectively.

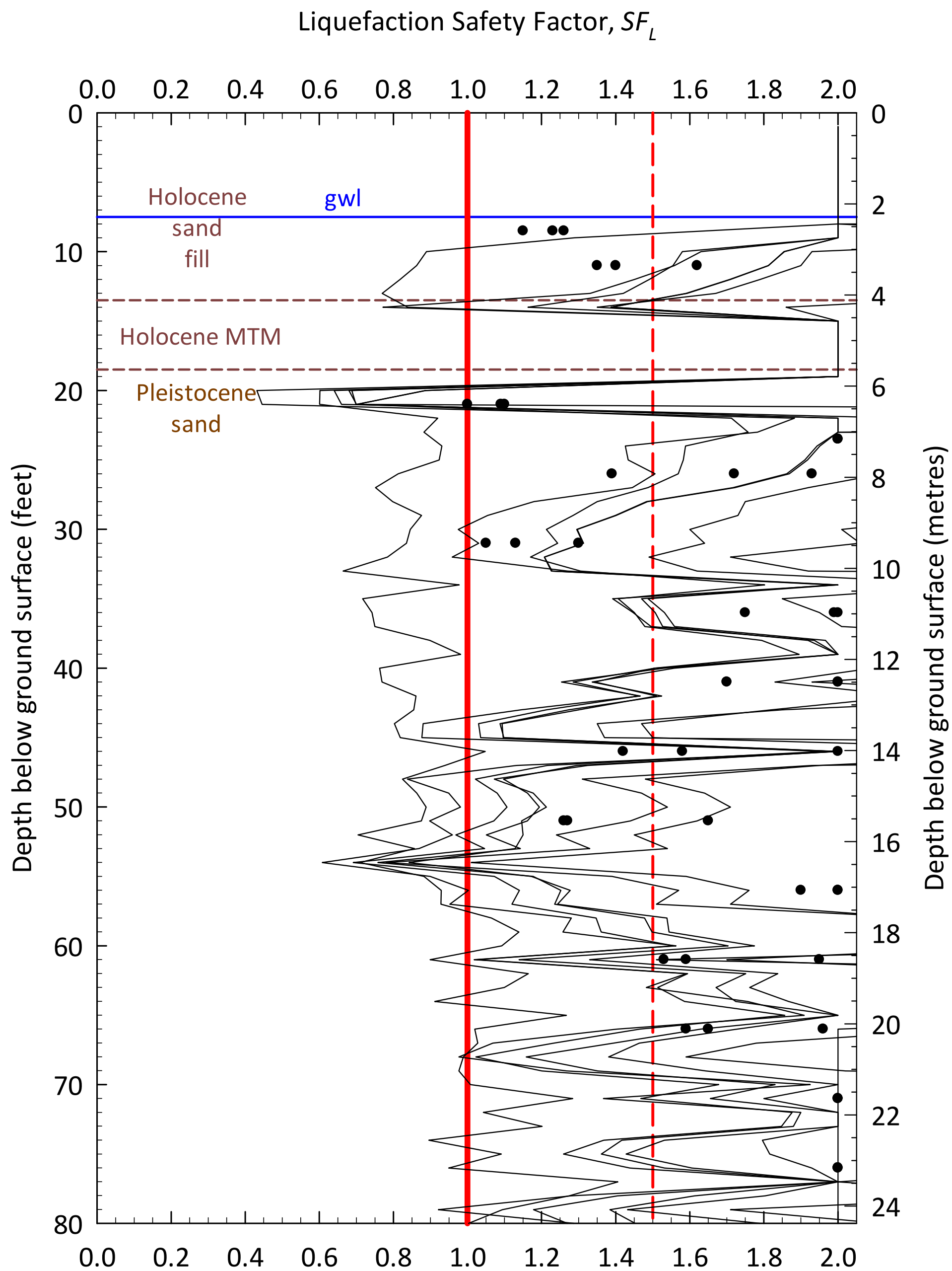

Figure 28. Liquefaction Assessment:
Present Conditions/No Aging/Summary of Results from All Versions.

The potential for liquefaction between these two depths is less clear. Only the recent (2014) CPT-based Boulanger-Idriss/2014 version is unequivocal about the likelihood of liquefaction potential within this zone.

Before closing the discussion of liquefaction completely, it is of interest to address the soil-aging issue, at least to some degree. As discussed previously, the age-related situation at JFKIA is more complicated than at the sites considered by Olson et al. (2001), Gassman et al. (2004), and Leon et al. (2006) that were cited previously. In the cases considered by these authors, significant changes in vertical effective stress due to filling or excavation by human activity were not an issue. Therefore, the issue of aging effects as they potentially affect liquefaction assessments at JFKIA needs to be the subject of a study well beyond the capability and intent of this paper.

The primary issue regarding aging at JFKIA is that the stratum where liquefaction is of greatest concern is of Late Pleistocene geological age, of the order of perhaps 10,000-years old (10 ka bp). However, if portions of this stratum liquefied one or more times during past earthquakes (perhaps as recently as the 1884 event that is believed to have been epicentered just south of JFKIA) then this would give these soils a much younger behavioral age.

Two sets of aging-related analyses were performed. Both used only the five CPT-based methodologies in *CLiq* that were used previously.

To begin with, in order to get a first-order assessment of the liquefaction potential that might have existed prior to construction of JFKIA, a feature of *CLiq* that allows excavation and a change in groundwater level to be analyzed was used. The results of this suite of analyses are shown in Figure 29. Note that no aging correction was applied to any of the analyses shown in this figure.

The results in this figure indicate that liquefaction within the upper portion of the Pleistocene sand stratum was quite likely and may have occurred down to a depth (relative to the current ground surface) somewhat below 50 feet (16 m). This conclusion seems plausible to the writer as it could explain why the upper portion of this stratum exhibits indications of overconsolidation (Figure 14) and relative densification (Figures 16 and 17) at the present time. It is conceivable that one or more prior seismic events caused the upper portion of this stratum to liquefy with a concomitant post-liquefaction increase in soil density.

For the sake of completeness, it should be noted that an exception to this overconsolidation and densification exists currently within the uppermost portion of the Pleistocene sand stratum, i.e. the first few feet (one metre) directly beneath the Holocene MTM. This relatively thin zone of soil is quite loose and as shown in Figure 28 highly likely to liquefy in a future significant earthquake. The most likely explanation for this is that this thin zone of sand was disturbed and reworked by wave or perhaps even winter-ice action during the Holocene, contemporaneous with the deposition of the Holocene MTM soils. In fact split-spoon samples of the soil within this thin zone of the uppermost Pleistocene contain minor amounts of organic clays indicating natural mixing of soil particles ('marbling') from two very different geologic times.

The influence of soil aging on the liquefaction assessment of the pre-construction soil profile was investigated in a very limited fashion by performing analyses using the Boulanger-Idriss/2014 version of the cyclic-stress method both without (already shown in Figure 29) and with an aging correction factor. An aging correction factor[80] = 1.3 was used. This factor was based on the Kulhawy-Mayne empirical equation for aging mentioned in Olson et al. (2001), Gassman et al. (2004), and Leon et al. (2006) and an assumed geological age of 10 ka.

The results of this comparison are shown in Figure 30. Significant liquefaction of the upper portion of the Pleistocene sand bearing stratum is still indicated but slightly less deep which actually improves the correlation with the aforementioned observed overconsolidation and densification.

[80] Implementation of an aging correction factor is another feature of *CLiq*.

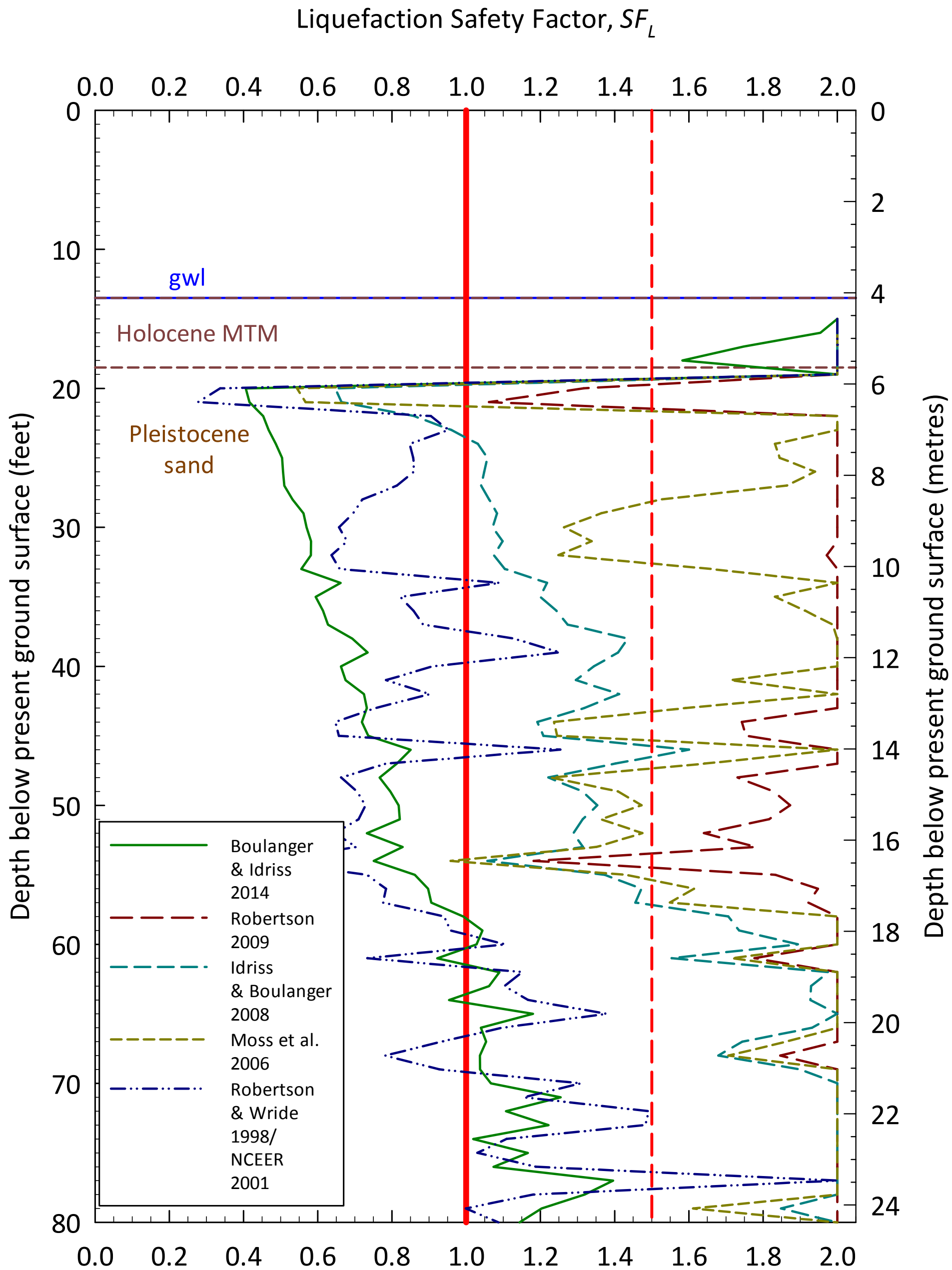

Figure 29. Liquefaction Assessment: Pre-Construction Conditions/No Aging/ CPT No. CTP-2 Data - Overall Comparison of Versions.

The conclusion drawn from this limited assessment of pre-construction liquefaction potential is that given the known history of at least two seismic events in the NYC metropolitan area with M = 5+, including the most-recent event in 1884 that is believed to have been epicentered not far from JFKIA, liquefaction of the upper portion of the Pleistocene sand stratum (formally the Upper Glacial Aquifer) beneath JFKIA could have occurred as recently as 1884. This would make these soils 'young' in terms of their behavioral age for the purposes of liquefaction assessment under current site conditions despite the fact that their geological age is much greater.

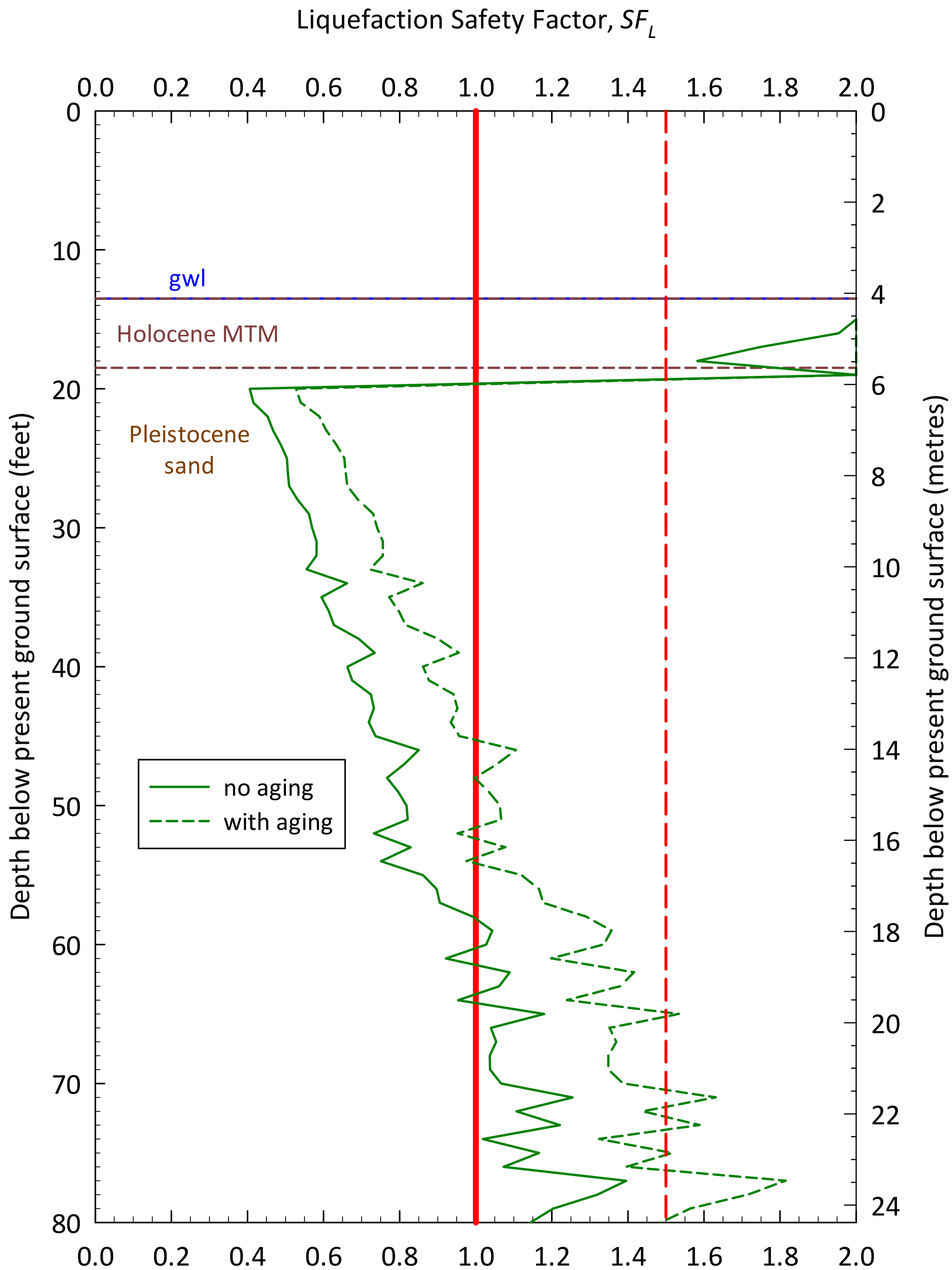

Figure 30. Liquefaction Assessment: Pre-Construction Conditions/Influence of Aging/ CPT No. CTP-2 Data - Boulanger-Idriss/2014 Version.

Nevertheless, it is of interest to re-analyze the current conditions using the 1.3 aging-correction factor to at least see what influence this has on the calculated results. This was done only for the five CPT-based versions of the cyclic-stress method.

To begin with, Figure 31 compares the no-aging versus aging results for the Boulanger-Idriss/2014 version as this was the version that produced the most conservative forecast of likely liquefaction for the no-aging case shown in Figure 21. As was the case with the pre-construction conditions (Figure 30), the difference is not insignificant although much of the upper portion of the Pleistocene sand stratum would still likely liquefy.

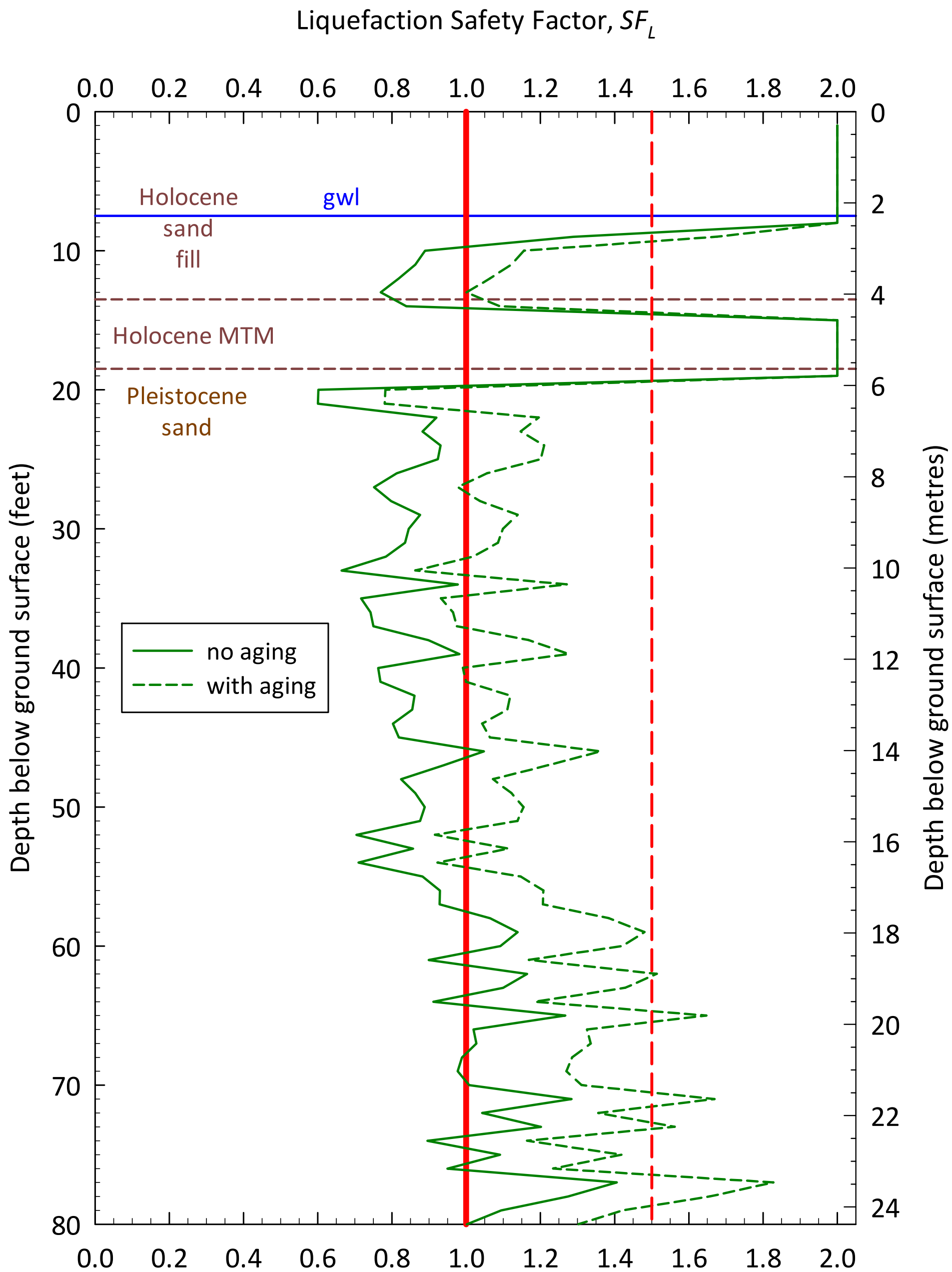

Figure 31. Liquefaction Assessment: Present Conditions/Influence of Aging/ CPT No. CTP-2 Data - Boulanger-Idriss/2014 Version.

Figure 32 shows the results for all five versions of the cyclic-stress method available in *CLiq*. The results in this figure can be compared to those in Figure 21 that shows the results for the original analyses that neglected any aging correction. As can be seen, even with an aging allowance there is still relatively broad agreement of the probability of liquefaction occurring near the very top of the Pleistocene sand stratum as well somewhat below depth BGS of 50 feet (16 m).

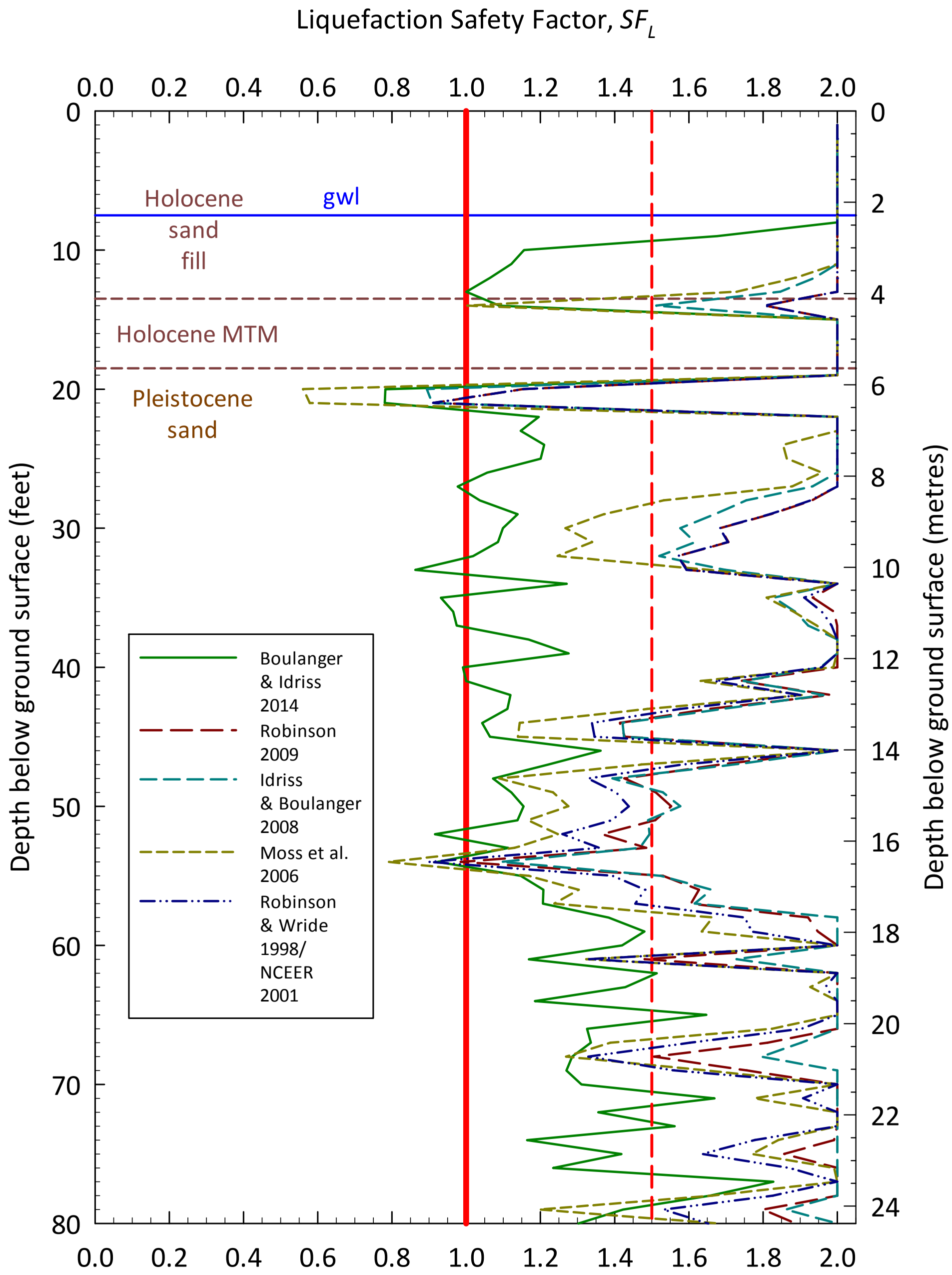

Figure 32. Liquefaction Assessment: Present Conditions/With Aging/ CPT No. CTP-2 Data - Overall Comparison of Versions.

Summary and Conclusions

To close out this lengthy discussion of liquefaction potential at JFKIA, although the analyses performed for this paper were limited in scope they were, in the writer's opinion, sufficient to indicate that there is a possibility that past seismic activity caused liquefaction of the Upper Glacial Aquifer that is the current bearing stratum for all deep foundations at JFKIA at some time prior to airport construction, perhaps as recently as 1884. Furthermore, it is likely that portions of this stratum may liquefy again in a relatively large earthquake.

However, the plethora of analytical methods that has evolved in the past 20 years since workshops were convened in the 1990s to resolve the then-state of "chaos" as Youd put it combined with collateral issues such as aging effects have, in the writer's opinion, created an even larger state of chaos at the present time. There is considerable burden on practicing engineers to select a version of the cyclic-stress method, or perhaps some number of versions, in which to place their trust as to the accuracy of the calculate outcome.

Deep-Foundation Capacity

Overview

From the very beginning, driven piles to support axial-compressive forces have been the deep foundation of choice at JFKIA. Although alternative deep-foundation technologies based on drilling as opposed to driving existed in the 1940s when construction at JFKIA first began, the subsurface conditions there have always been viewed as classical friction-pile conditions and thus dealt with accordingly. However, as will be discussed in a subsequent section dealing with pile types the need in recent years to consider liquefaction has profoundly impacted not only the types of driven piles considered for use but also resulted in rethinking whether some type of drilled deep-foundation alternative might be more cost effective.

Another consequence of the now-requisite seismic design is the relatively significant horizontal forces (lateral loads) brought down to the pile caps that must be resisted by whatever type of deep-foundation element is used. Although this requirement does not appear to have had primary influence on deep-foundation selection at JFKIA there is at least one significant issue related to accommodating such loads that is discussed subsequently as it has broad impact on practice elsewhere.

The discussion in this section of the paper will focus on axial-compressive geotechnical ultimate resistance (capacity). As is well known, there are two broad conceptual approaches for estimating this for a driven pile:

- the <u>dynamic</u> approach in which it is assumed that the pile's physical resistance to energy input from a pile-driving hammer during driving[81] can be correlated with long-term, post-driving geotechnical resistance under static and/or quasi-static loads; and

- the <u>static</u> approach based on applying soil-mechanics principles to a pile that is assumed already embedded in the ground.

The dynamic approach is the older of the two so is discussed first.

Dynamic Approach

The dynamic approach dates back at least to the mid-19th century (Chellis 1961, Likins et al. 2012) thus pre-dating modern soil mechanics by many decades and the static

[81] A parameter often used as the metric for this purpose is the pile set which is defined as the magnitude of pile penetration into the ground (dimensions of length and typically expressed in units of either inches or millimetres) either per hammer blow or averaged over some arbitrary number of blows. Alternatively and more commonly in U.S. practice, driving resistance, which is the reciprocal of set, is used as the metric (with dimensions of length^{-1}) and typically expressed as some number of blows per foot, inch, metre, or millimetre of pile penetration.

approach by about a century. In its lifetime the dynamic approach has gone through two very distinct evolutionary episodes.

Initial efforts in the 19th century were more-or-less based on pure physics and involved two different concepts. At first, circa mid-century, a relatively simplistic physical concept (work done by pile hammer = work from pile resistance) was assumed. In this case work was defined in the classical physics sense as simply the product of a force times the displacement of some object associated with that force. So the work done by the pile hammer was simply the weight of the hammer ram (a simple drop-weight in that era) times the distance it fell, both of which quantities were always known. The work done by the pile was simply its geotechnical resistance times the set of the pile under a hammer blow. Although the former was unknown the latter could be measured so one was left with one equation with one unknown so calculating the pile resistance was straightforward.

More involved was the second, later concept employed which was the physics equation for impact of rigid bodies in motion. This concept gained quite a bit of traction around the cusp of the 20th century to the point that it had a lasting, continuing (into the 21st century) effect on practice in the form of the well-known, so-called dynamic formulas or pile-driving formulas such as the *Engineering News* formula.

As a group, these formulas have long been recognized as 'bad science' that should have been abandoned decades ago simply because they do not capture the true physics of installing piles using traditional impact driving. However, their use has endured in practice to the present due to their elegant simplicity that geotechnical engineering practitioners and others who use them seem to find irresistibly appealing to the extent that they ignore scientific fact as to their worthlessness.

As is now well known, the inherent deficiency and concomitant fatal flaw of all early attempts of the dynamic approach was the assumption that a pile behaves as a rigid body under impact driving. The reality is that any pile subjected to impact driving exhibits relatively significant axial flexibility (compressibility) during driving that can be modeled using the theory of one-dimensional (1-D) stress-wave transmission through a linear-elastic rod. This is commonly referred to as the wave equation. The seminal work of Smith (1960) is generally cited and acknowledged as the watershed event in development of the wave equation as a physical and mathematical model for pile driving. Smith's work ultimately led to the broader use of wave mechanics for assessing deep foundations installed by other means such as drilling.

There was a certain element of fortuitous timing as Smith's work was published at the dawn of the computational revolution brought on by the digital computer as discussed previously. This was significant as a wave-equation analysis of a pile is not feasible without a numerical solution using a digital computer. As the 1960s progressed, large mainframe computers became available commercially and were quickly acquired by major academic institutions among others.

During the same decade (1960s), research was conducted at the Texas Transportation Institute (TTI) of Texas A&M University to develop software that practitioners could use to perform wave-equation analyses. This research culminated in a comprehensive report by Lowery et al. (1969) that contained the Fortran IV code for what later was, and nowadays is still referred to as, the TTI version (or some similar term) of the wave-equation program to distinguish it from subsequent programs developed and sold by others.

The writer has first-hand knowledge that the PANYNJ was at the forefront of computational capability with regard to use of the wave equation in general and for project-specific use at JFKIA in particular. The TTI program had been acquired by the PANYNJ at least by mid-1972 and was used extensively during 1972-3 to evaluate test piles (sometimes referred to as indicator piles in those years within the PANYNJ) driven as part of the *IAB-STRAP* project[82] although the writer has no recollection if this was the first project on which PANYNJ engineers used the wave equation.

In any event, it is not known to the writer to what extent wave-equation analyses have continued to be used by various stakeholders (design engineers and contractors) involved in driven-pile installation at JFKIA in the 40-plus years since then. However, other wave-related analytical technologies (by happenstance used for the first time at JFKIA and by the PANYNJ for the same 1972-3 study) have seen ongoing usage. These will be discussed in the section dealing with technical-needs fulfillment.

Static Approach: Concepts and Theory

As noted previously, the start of initial construction of what would eventually become JFKIA was contemporaneous with the publication of Terzaghi's seminal English-language textbook *Theoretical Soil Mechanics* in 1943 in which he stated (Page 137):

> *"Since the bearing capacity of piles cannot yet be computed on the basis of the results of soil tests performed in the laboratory we are still obliged either to estimate this value on the basis of local experience or else to determine it directly in the field by loading a test pile to the point of failure."*

What Terzaghi was referring to as not being possible to do at that time is related to what we would now call the static approach for calculating the axial-compressive geotechnical ultimate resistance of a deep-foundation element. While laboratory testing of soil samples is still impractical for deep foundations in coarse-grain soils there have been enormous strides in in-situ testing in all types of soil as well as soil mechanics theory that now allow what was infeasible circa 1943 to be performed with acceptable accuracy on a routine basis. Because the advances in geomechanics relative to the static method of axial-resistance calculation are many and varied, it is useful to divide the discussion between developments related to in-situ testing and those related to theoretical soil mechanics advances.

In-situ testing in general, and its use at JFKIA in particular, have already been discussed in detail. In general, the outcomes (i.e. the measured parameter(s) unique to the device) from any in-situ testing program can be used in two broadly different, distinct ways for estimating the axial-compressive geotechnical ultimate resistance of driven piles (or deep foundations in general for that matter). They can be correlated:

- with fundamental soil properties (index properties, stress-state, stiffness, strength) with the resulting soil properties being used in some theoretical analytical process; or

- directly with the traditional capacity mechanisms (shaft and toe resistances) via empirical relationships.

As with many technical issues in civil engineering, there are pros and cons, pluses and minuses as well as proponents of each conceptual approach. It is not the intent of this paper to address this issue in detail or take a position or sides other than to note that in practice the two approaches are not and should not be mutually exclusive. In practice one

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[82] The various PANYNJ test-pile programs at JFKIA of which the writer is aware are discussed in detail in Horvath and Trochalides (2004) as well as in a subsequent section of this report that deals with pile types.

can certainly evaluate resistance using two or more distinctly different methods in order to develop a range of estimates in an attempt to quantify the reliability and uncertainty of calculated resistances.

That having been said, it is of relevance to note that one advantage of the two-step process of first correlating measured parameters from in-situ testing with fundamental soil properties and then using those properties in a theory-based analysis is that the effect of and interaction between various soil properties is clearly seen. This is noted because the writer has used this to advantage over the past decade of research related to the capacity of driven piles at JFKIA (Horvath 2002, 2003a, 2003b, 2003c, 2004; Horvath and Trochalides 2004; Horvath et al. 2004a, 2004b).

This, then, leads to the discussion of advances in theoretical soil mechanics with respect to estimating deep-foundation axial-compressive geotechnical ultimate resistance subsequent to Terzaghi's expressed opinions in 1943. In the writer's opinion, the single most significant development in this regard has been the formal recognition (but, in the writer's opinion, not nearly to the extent it should) of the unique niche held by tapered piles that are defined here as any pile with a lengthwise variation in perimeter, whether continuous, step-wise, or partial.

One might argue that humans have always benefitted from, if not explicitly recognized the benefit, of tapered piles as timber piles are naturally continuously tapered. Thus tapered piles in the form of timber piles have been used since antiquity. However, this was more out of necessity (there were no alternatives until PCC and steel were developed and evolved as construction materials) than scientific insight or genius. Nevertheless, U.S. patent records make it clear that the benefit of tapered piles must have been recognized and appreciated to some degree based on the number of patents filed around the turn of the 20th century, i.e. decades before the development and promulgation of modern soil mechanics and Terzaghi's 1943 book. These patents were for both continuously-tapered as well as step-tapered piles composed of various combinations of PCC and steel (Horvath 2003b).

In any event, one post-1943 landmark in the recognition of the benefit of tapered piles is the study of driven piles documented in Peck (1958) wherein he stated:

> *"...it is obvious from an inspection of Figure...that taper has a beneficial influence on the capacity of piles in sand...it would appear reasonable to conclude that a taper of 1 percent or more is likely to increase the capacity of a pile, for a given length of embedment, between 11/2 and 21/2 times."*

Attempts to formally quantify this benefit using modern soil mechanics concepts and principles can be traced back to Nordlund (1963) who deserves credit for recognizing that the physical mechanism that gives tapered piles this capacity benefit is fundamentally different from the traditional capacity mechanisms of shaft- and toe-resistance that all deep-foundation elements possess when loaded in axial compression. Specifically and as illustrated in Figure 33, he recognized that as the tapered portion of a pile displaces downward under an externally-applied vertical force (either initially during driving or subsequently during loading) past an arbitrary, spatially-fixed horizontal plane (*x-x′* in the figure) within the ground, from the perspective of that plane it appears as though the pile diameter is increasing. This apparent increase in pile diameter has the effect of the pile expanding and pushing horizontally outward on the adjacent soil thereby increasing the horizontal stresses within that soil. The overall physical mechanism at work here is a very basic, simple one in physics, that of a wedge whereby simple geometry translates vertical force and concomitant displacement into horizontal displacement and concomitant force.

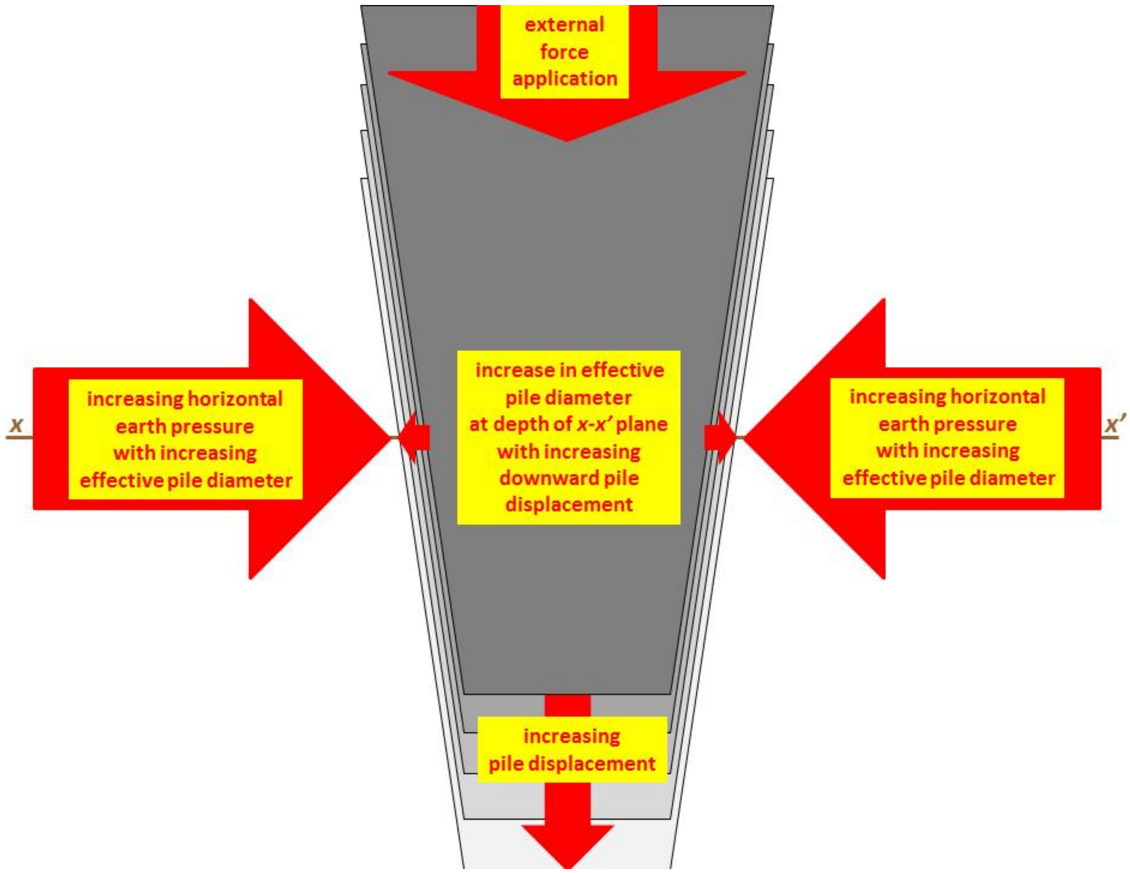

Figure 33. Tapered Pile Behavioral Mechanism.

Nordlund developed a theoretical solution for pile capacity based on this model but it was hampered by limits in the state of soil-mechanics knowledge at the time and the resulting theoretical model he used to develop this solution. Specifically, he envisioned the effective horizontal displacement of the pile shaft as being modeled by passive earth-pressure theory, with lateral earth-pressure coefficients for the adjacent soil developed accordingly. The inherent problem with this is that a two-dimensional (2-D) model was used for what is clearly a three-dimensional (3-D) problem.

Although this physical mechanism is clearly different from the traditional deep-foundation mechanism of shaft and toe resistance, Nordlund did not analytically treat it as a distinct capacity mechanism but rather a subset or modification of the traditional shaft-resistance mechanism. The net outcome is that Nordlund's solution treats tapered piles as simply having increased shaft resistance compared to non-tapered piles. Nordlund did not consider the toe-resistance mechanism and concomitant toe resistance of tapered piles to be affected by the taper.

While Nordlund's work was a significant step forward in terms of analytically recognizing the benefit of tapered piles, the true nature of the tapered-pile behavior under axial-compressive loading was not correctly identified and explored until the work by Kodikara about 30 years later (Kodikara and Moore 1993). Kodikara modeled the effective increase in pile diameter due to downward displacement of the pile as cylindrical-cavity expansion, making use of the extensive research into cavity-expansion theories that had occurred in the years subsequent to Nordlund's paper. While cavity expansion is broadly, conceptually similar to developing passive earth pressures as Nordlund modeled the process, using cylindrical cavity-expansion theory, which is inherently a 3-D theory, more accurately captures what is going on within the soil adjacent to the pile.

In addition, and importantly in the writer's opinion, Kodikara identified this cylindrical-cavity expansion as a pile-capacity mechanism physically different and thus distinct from shaft resistance. With cavity expansion, the horizontal earth pressures within the soil at a spatially-fixed point along the pile shaft are increasing (within limits) as the volume occupied by the pile at that point increases with increasing downward displacement of the pile as depicted in Figure 33. On the other hand, for a non-tapered pile with a constant perimeter the horizontal stresses at the same spatially-fixed point would be expected to remain more or less constant as the pile displaced downward.

For the sake of completeness, it should be noted that other, alternative treatments have been proposed in recent years for dealing with tapered piles. The primary one with which the writer is familiar was suggested by Fellenius (2014) who stated that the continuously-tapered portion of a pile could be modeled as a step-wise variation in pile perimeter with depth[83], with the length of each artificial step being arbitrary. The resulting fictitious annular area at the bottom of each artificial step is then analyzed assuming a toe-resistance mechanism that provides a pseudo-toe resistance that is additive to the shaft-resistance for that artificial pile segment. Thus the capacity along the shaft of a tapered pile is artificially decomposed and analyzed as the sum of a constant-perimeter shaft resistance plus a pseudo-toe resistance.

While this analytical model may be intuitively pleasing, there is no published evidence that the writer has seen to demonstrate that it provides accurate estimates of tapered-pile resistance. Rather, this approach seems to be simply a way to force a tapered pile to fit into the well-established framework (specifically, Fellenius' well-known β-method) that all deep foundations only have shaft- and toe-resistance and thus avoid having to develop or accept the necessary and proper elements of a true third-capacity mechanism.

In any event, Kodikara's seminal work clearly demonstrated that, in principle, there are three potential mechanisms by which any deep-foundation element develops axial-compressive geotechnical resistance:

- the traditional mechanisms of shaft- and toe-resistance plus
- cylindrical-cavity expansion.

Unfortunately, this understanding has failed to gain traction and recognition among both practitioners and researchers over the past 20-plus years since the publication of Kodikara's work. In fact, even Nordlund's basic concepts behind the behavior of tapered piles are not as well-known as they should be given their 50-plus years of existence. In the writer's opinion this is most unfortunate as understanding both the technical benefit of, as well as the true capacity mechanism behind, tapered piles could be beneficial for both optimizing pile designs as well as improving the accuracy of capacity calculation which has long been problematic for tapered piles.

Of course ignorance about this third-capacity mechanism also permeates textbooks and thus basic civil-engineering education which means that not only current but future generations of foundation engineers continue to underappreciate (and even be totally ignorant of) and thus underutilize tapered piles in practice. Furthermore, unless and until the third capacity mechanism of cylindrical-cavity expansion is recognized, the traditional wave-equation model, which only accounts for shaft- and toe-resistance to driving, will

[83] This model mimics the actual physical appearance of the old *Raymond Step-Taper*® pile.

continue to inadequately replicate what happens during the driving of tapered piles. As will be discussed subsequently in the section on technical-needs fulfillment, the inability of current wave-based analytical techniques to properly capture and model the third capacity mechanism of cylindrical-cavity expansion when driving tapered piles impacts field-based dynamic measurements as well.

The increase in horizontal stresses within the soil adjacent to a tapered pile is a subset of the larger issue of how horizontal stresses in the ground affect the shaft resistance of all types of deep foundations. From the very beginning of when static-capacity analytical methods began to be developed and evolve in the 1950s it has been recognized that the traditional shaft-resistance capacity mechanism of all deep-foundation elements is linearly proportional to the horizontal stress in the soil adjacent to the element.

Unfortunately, part of the evolution of the profession's understanding of this capacity mechanism has taken some wrong turns and had dead-ends along the way, perhaps the most significant being the now-debunked fallacy of a 'critical depth' and concomitant 'limiting pressure' (Kulhawy 1984, 1991, 1996; Fellenius and Altaee 1995, 1996) that was once promulgated in several venues, including respected textbooks on the subject (e.g. Poulos and Davis 1980). But it is now recognized that, in general:

$$\sigma'_h = K_h \cdot \sigma'_{vo} \qquad \textbf{(10)}$$

where:

σ'_h = horizontal effective stress at the interface between soil and deep-foundation shaft,
K_h = dimensionless coefficient of horizontal (lateral) earth pressure, and
σ'_{vo} = vertical effective overburden stress at a depth of interest along the deep-foundation shaft.

That K_h depends on the type of deep-foundation element, primarily its method of installation but also its geometry (in the case of driven piles) and composition, has long been known. However, in the writer's opinion one of the most significant outcomes of more-recent research is that the coefficient of lateral earth pressure at rest, K_o, prior to installation of the deep-foundation element is also an important, critical parameter (Kulhawy 1984, 1991). In fact, current thinking is to relate not K_h but the ratio K_h/K_o to the type of deep foundation, method of installation, etc. so that Equation 10 is better expressed as:

$$\sigma'_h = (K_h / K_o) \cdot K_o \cdot \sigma'_{vo} \qquad \textbf{(11)}$$

where all terms have been defined previously.

Note that Equation 11 indicates clearly that the stress-state in the ground prior to installation of any type of deep-fo*u*ndation element is a significant variable affecting the post-installation shaft resistance. In hindsight, this seems so obvious but prior to Kulhawy's published work this had not be considered explicitly in analytical methodologies for deep-foundation capacity and is still not in the mainstream of analytical practice even though this information has been available for three decades.

The direct impact that K_o has on shaft resistance is another reason why in-situ testing in general, and the CPT in particular, and the various analytical algorithms and empirical relationships for coarse-grain soil as discussed previously and illustrated in Figure 15 are such important aspects of modern practice on projects of any size.

It is worth noting that this direct influence that the existing, pre-installation K_o has on shaft resistance significantly impacts the popular (e.g. Hannigan et al. 1998, Horvath

2002, Fellenius 2014) β-method for calculating shaft resistance. This can be demonstrated as follows.

To begin with, the empirical, dimensionless parameter β is defined as:

$$r_s = \beta \cdot \sigma'_{vo} \tag{14}$$

where:

r_s is the unit shaft resistance and
σ'_{vo} = vertical effective overburden stress at a depth of interest along the pile shaft as before.

The fundamental definition of r_s is:

$$r_s = K_h \cdot \tan\delta \cdot \sigma'_{vo} \tag{15}$$

where δ is the pile-soil interface friction angle.

Equating Equations 14 and 15 then multiplying the net result by 1 (= K_o/K_o) as was done to produce Equation 11 from Equation 10 yields:

$$\beta = K_h \cdot \tan\delta = (K_h / K_o) \cdot K_o \cdot \tan\delta \tag{16}$$

which suggests that a better parameter than β alone with which to correlate observed unit shaft resistance for all types of deep foundations would be the ratio

$$\beta / K_o = (K_h / K_o) \cdot \tan\delta \tag{17}$$

as this would more accurately reflect the installation effects of a particular type of deep-foundation element relative to the pre-existing overburden stresses, i.e. the ratio K_h / K_o.

This concept was explored in a preliminary fashion by Horvath and Trochalides (2004) using the unit shaft resistance, r_s, obtained by calculation (the method used is discussed below) and what were felt to be the actual pre-driving K_o values estimated from site characterization for a variety of driven piles with different taper angles driven at JFKIA (the values were similar to those shown in Figure 15 for the CTA at JFKIA although many of the piles studied by Horvath and Trochalides (2004) were outside the CTA). The goal of this exercise was to see if there was a correlation between β and pile taper angle ω as one would expect from Nordlund's work.

Figure 34 is Figure 4 from Horvath and Trochalides (2004) and shows the initial attempt at correlating β and ω (the latter expressed in degrees of angle)[84]. Note that the ordinate in this figure (and Figure 35 subsequently) is a non-dimensional relative depth. This is defined here as the actual depth below the start of the in-ground tapered portion of

[84] It is of interest to note that defining a single-value of taper angle for a timber pile is inherently difficult and, ultimately, approximate. This is because this angle typically varies along the length of the pile as an artifact of nature. As an example using one of the five Southern Yellow Pine timber piles reflected in Figures 34 and 35 (designated No. LT10-172), the pile was 60 feet (18 m) long prior to driving. Prior to installation the diameter was measured at five points: head, toe, and quarter points. This produced average taper angles for each of four quarter-lengths of the pile as follows (going from head to toe): 0.15°, 0.18°, 0.27°, and 0.22° for an average of 0.21°. However, only 43 feet (13 m) of this pile was installed in the ground of which only about the lower 25 feet (8 m) was embedded in the Pleistocene sand bearing stratum. This means the average taper angle within the bearing stratum is probably closer to about 0.24° than the overall average of 0.21°.

the pile divided by the total length of the in-ground tapered portion of the pile. Thus the latter quantity would be the entire embedded portion of the pile for timber piles and just the lower tapered portion for *Monotube*- and *Tapertube*-brand steel piles. In all cases a relative depth = 1 represents the toe of the pile. In any event, if there is a trend of β with ω in this figure it is not readily apparent, being lost in the scatter of the data.

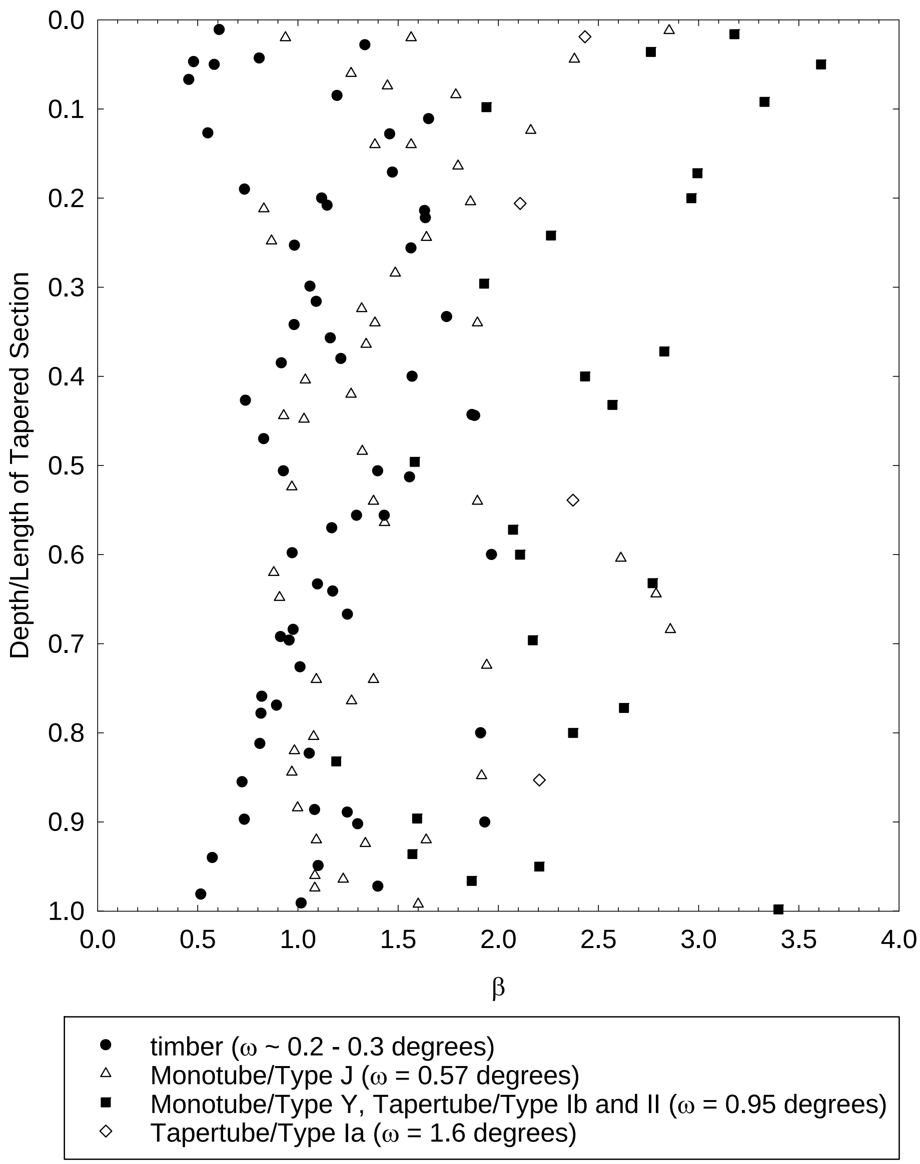

Figure 34. β-ω Relationship [from Horvath and Trochalides 2004].

As an aside, it is of interest to examine the inferred values of β compared to values that appear in the published literature (e.g. Fellenius 2014) that are presumably for piles with a constant perimeter with depth. The proportional increase in β due to taper (what can be termed a taper amplification factor) is as much as four- to five-fold which is the same order of magnitude increase one would expect from Nordlund's published work and about twice the taper benefit noted by Peck (1958).

In any event, in an effort to extract more-meaningful results from the data, the same parameters, β and ω, were then replotted using the ratio β/K_o as the abscissa. The results are shown in Figure 35 which is Figure 5 from Horvath and Trochalides (2004). In this case the trend of increasing β/K_o with increasing taper angle ω is obvious and much clearer. Other trends noted in this figure are:

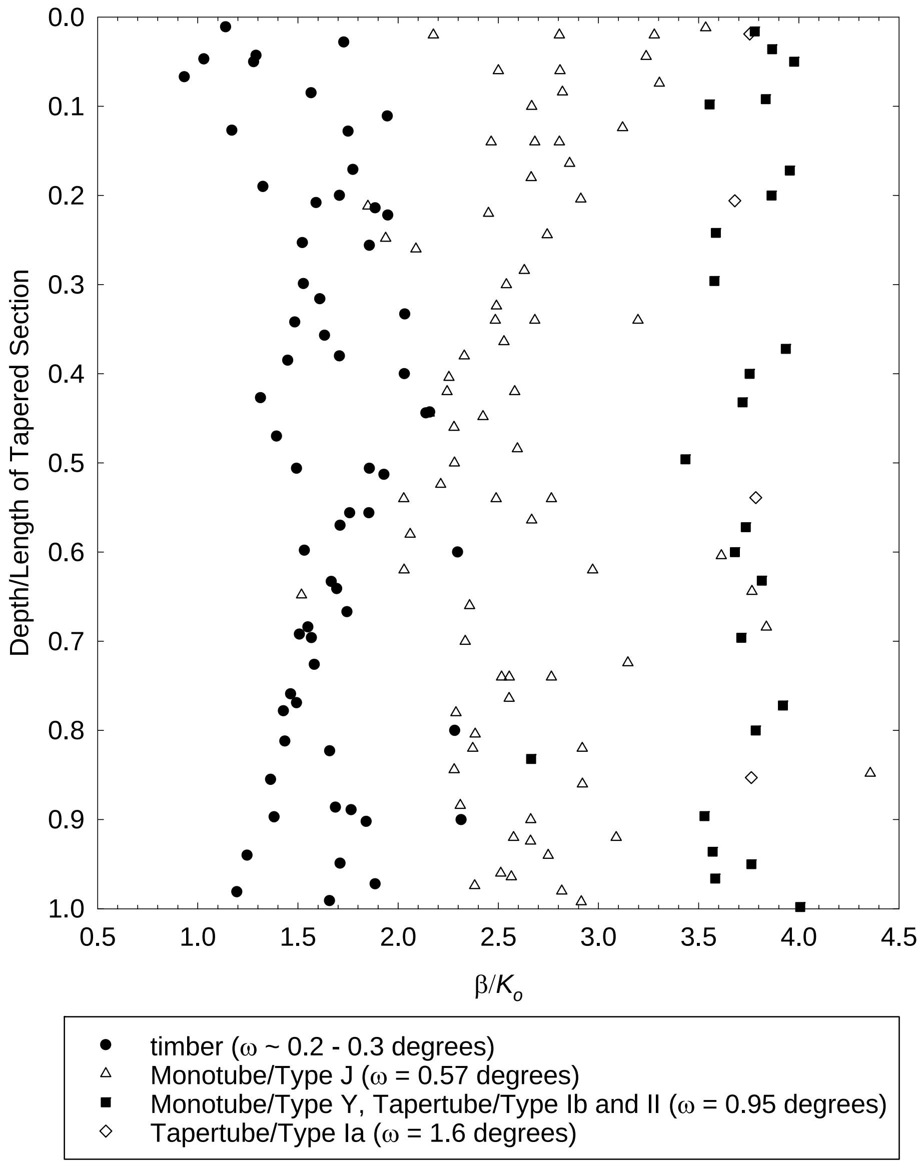

Figure 35. (β/K_o)-ω Relationship (from Horvath and Trochalides 2004).

- For a given taper angle, the β/K_o ratio is more or less constant along the tapered portion of a pile. This is something that would be expected based on cylindrical-cavity expansion theory.

- There appears to be a limiting effective value of taper angle beyond which there is no or at least no noticeable increase in benefit. This is also consistent with results in Nordlund

(1963) as well as cavity-expansion theories that always show a limiting pressure. In this case, there are essentially identical results for ω = 0.95° and 1.6° which may explain why steel piles with a manufactured taper angle greater than ω = 0.95° are not, for the most part, manufactured and used in practice. Note that the *Tapertube* Type 1a pile with ω = 1.6° shown in Figures 34 and 35 was an experimental prototype that was not used for the subsequent *Tapertube* Type 2 production piles that all used the ω = 0.95° of the other experimental prototype, *Tapertube* Type 1b.

As a final illustration of the benefit of taper, the data in Figure 35 were used to create Figure 36 that shows the variation of the ratio K_h/K_o as a function of taper angle. This is another way of illustrating the taper amplification factor relative to the baseline no-taper (constant-perimeter) case. The range in K_h/K_o for the baseline ω = 0° (constant-perimeter pile) is from Kulhawy (1984) and the range in ω for timber piles is also shown. The writer used the plotting software with which this figure was created (*SigmaPlot for Windows Versions 12.0*) to generate two best-fit curves to the data:

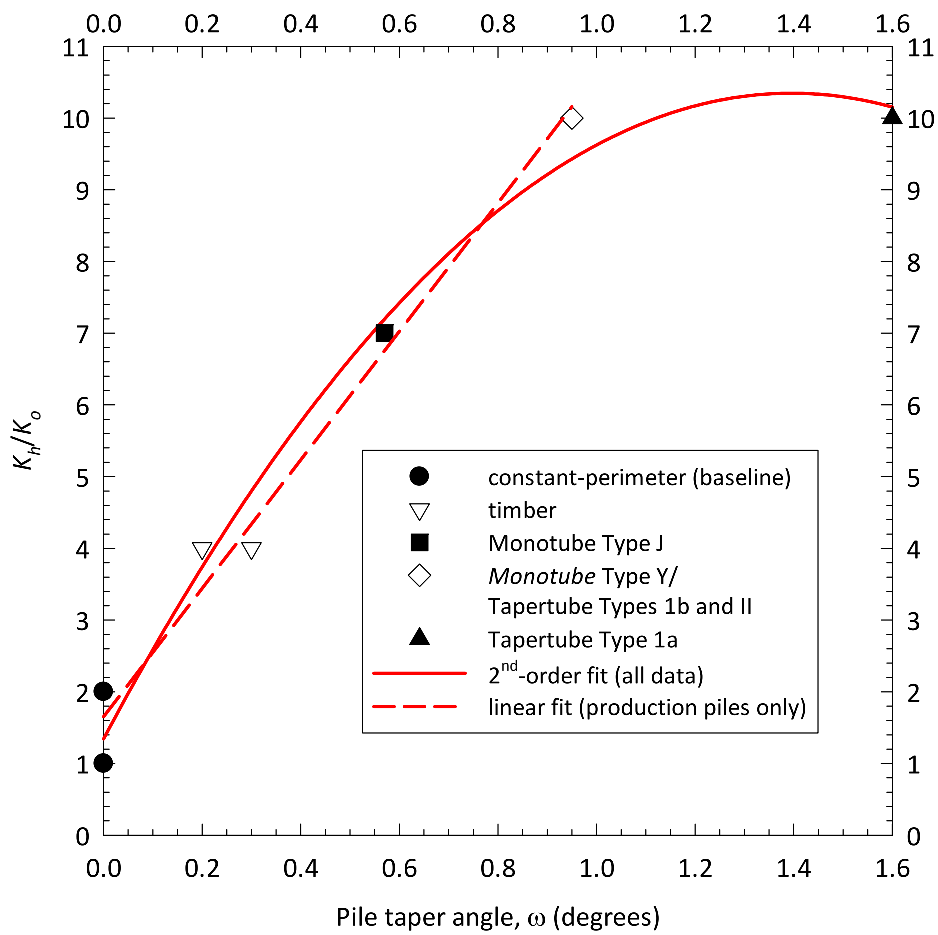

Figure 36. (K_h/K_o)-ω Relationship for Horvath and Trochalides (2004) Data.

- a second-order polynomial (solid red curve) to fit all data and
- a linear fit (dashed red line) to just fit the taper angles used for production piles in practice.

These two curves indicate that:

- At least as a first-order approximation, the taper amplification factor for tapered piles likely to be encountered in practice (at JFKIA at least) is linearly related to taper angle.

- There appears to be an upper-bound of approximately 1° of taper beyond which there is no or minimal additional benefit of taper. The taper amplification factor associated with this upper bound is approximately five. Stated another way,. all things being equal a pile with approximately 1° of taper angle will have a horizontal earth pressure acting along the shaft that is about five times that of a constant-perimeter pile in the same soil conditions.

Considering next the issue of toe resistance, this has long been a troubling calculation to perform accurately for several reasons. To begin with, there is the issue of deep foundations bearing on or in bedrock versus soils. In many cases involving the former case structural considerations involving the deep-foundation material control, not geotechnical considerations.

Even for deep foundations bearing completely in soil, not all deep-foundation elements loaded in axial compression exhibit a 'plunging' type of load-settlement behavior (i.e. effectively unlimited settlement at more or less constant load indicating a clearly-defined ultimate geotechnical resistance) that would be indicative of a classical bearing-capacity failure of any type of foundation element. Rather, the overall behavior is one of increasing capacity with increasing settlement at various rates. In fact, Fellenius (1999b, 2014) has gone so far as to suggest that the traditional bearing-capacity mechanism simply does not develop or exist for deep foundations bearing entirely in soil and is, therefore, an irrelevant, meaningless calculation for deep foundations in general and driven piles in particular. Fellenius goes on to argue that settlement is and should always be the controlling issue.

Setting aside this broader issue, even if one follows the more conventional, traditional approach of calculating a bearing capacity at the pile toe there is ample evidence that the traditional solutions used for shallow foundations grossly overestimate the bearing capacity. The reason why was clearly explained by Kulhawy (1984) who noted that an underlying assumption used to develop all traditional bearing-capacity solutions, i.e. that the soil can be modeled as an idealized rigid-plastic material where displacements and/or displacements and deformations of the soil up to the point of failure can be ignored, that are reasonable for most shallow-foundation applications is always incorrect for deep foundations. This means that soil compressibility (or its inverse, rigidity) must always be considered explicitly when evaluating the bearing capacity of deep foundations.

This is most directly accomplished using Vesic's rigidity factors although this adds a layer of significant complexity to the calculations not present with the basic bearing capacity that ignores soil-compressibility effects. Nevertheless, the writer has found the compressibility/rigidity correction reasonably straightforward to implement, especially when detailed site characterization based on in-situ test data has been performed.

For the sake of completeness, it is worth noting that there are other conceptual approaches of potential use to estimate the toe resistance of deep-foundation elements. These typically involve either some purely empirical correlation with one or more in-situ test parameters or some form of cavity expansion, usually spherical cavities which is an extension of earlier suggestions by Vesic (1972).

One example of work related to spherical cavity expansion is that of Yu and Houlsby (1991). The writer has performed some preliminary assessment of Yu and Houlsby's work for this purpose and found that it has complexities and concomitant implementation issues that suggest this would not be a straightforward methodology to use in routine practice.

More recently, the writer has considered (at least in concept) the possibility that cylindrical cavity-expansion theory may actually be more appropriate to use for modeling toe resistance. This is based on research for modeling CPT tip resistance, q_c, that shows the physical failure mechanism that develops at and below the tip of the cone penetrometer is better approximated as cylindrical cavity expansion as opposed to spherical cavity expansion as assumed historically (Salgado et al. 1997, Salgado and Prezzi 2007).

Note, however, that the cylindrical cavity-expansion solution that would be used for toe resistance is not the same as that used by Kodikara to model the third capacity mechanism for tapered piles. This is because the former is for a cavity that does not exist initially but is created and then expanded by the toe of the deep-foundation element whereas the latter is for a cavity that exists initially and is then expanded further as the deep-foundation element displaces downward.

The writer has attempted to utilize some of the research developments that are outlined above to improve the state of practice for estimating the axial-compressive geotechnical ultimate resistance of tapered piles in coarse-grain soil. The extensive database of subsurface and pile data for JFKIA, portions of which were made available to the writer many years ago for academic-instruction and -research purposes, have figured prominently in this work.

The writer's initial efforts at developing an improved analytical method for tapered piles had their origin in the late 1980s but work did not begin in earnest until circa 2000. This initial work led to the publication of Horvath (2002) that contained a very detailed assessment of pile capacities using different analytical methods, including one developed by the writer as discussed subsequently. There have been several published summaries of this work (Horvath 2003b, 2003c; Horvath and Trochalides 2004; Horvath et al. 2004a, 2004b) as well as updates (Horvath 2003a, 2004), with the overall effort still being a work-in-progress.

The writer's research indicated that Kodikara's theoretical solution for the third capacity mechanism based on cylindrical cavity expansion is quite complex to use in terms of implementing it into a numerical algorithm to be solved by computer. As a result, it is the writer's opinion that it will take further development to bring Kodikara's work to the point where it can be used routinely in practice as part of an overall resistance-calculation algorithm.

In the interim, the writer modified and extended Nordlund's concepts to incorporate the influence of K_o as reflected in Equation 11. Evaluating K_o in routine practice has been rendered feasible to do on even the smallest projects where nothing but SPT *N*-values are available by virtue of the significant developments in site characterization based on in-situ testing as discussed in detail earlier in this paper and shown in Figure 15.

Other modern soil-mechanics concepts that were utilized in developing the shaft-resistance component of the writer's Modified Nordlund analytical methodology include consideration of:

- the overall strain-softening stress-strain behavior of soil that results in a soil having both peak and constant-volume (critical-state) Mohr-Coulomb angles of internal friction, ϕ; and
- the significance that dilatancy can have on the operative value of the peak friction angle, ϕ_{peak}, and, as a result, the observed value of unit shaft resistance r_s, a concept explored by both Kulhawy (1984) and Houlsby (1991).

An essential aspect in the development of any new analytical method for the geotechnical axial-compressive resistance of deep foundations is to compare results calculated using the new methodology with ground-truth, i.e. actual geotechnical ultimate resistances. Historically, the actual geotechnical ultimate resistances used to assess various analytical methodologies developed over the years were and are almost always trivially taken to be a given requiring little or no discussion as to how they were determined. However, the current state of knowledge clearly indicates that this should never be the case in the future for several reasons.

The primary one involves the traditional static load test. As noted previously, the load-settlement behavior of a deep-foundation element bearing entirely within soil does not always exhibit a well-defined load at which plunging behavior, which would unambiguously define an ultimate failure, develops. This is due primarily to the nature of the toe resistance and is the underlying basis of the extreme position taken by Fellenius (1999b, 2014) that bearing failure in the traditional, classical sense simply never occurs. In fact, it is the very ill-defined nature of a failure load that led to the development of many different empirical methods for estimating same in load tests as discussed in detail by Fellenius (1990, 2001) and noted in Horvath (2002). So even for a single load-settlement curve produced by an actual load test of some kind there will generally be a surprisingly large range of interpretations of what constitutes the failure load, with some methods even producing load magnitudes greater than actually applied in a test.

As an aside, it is worth noting that the widely (mis/over)used Davisson Offset Limit Method (Davisson 1970, 1972) for defining failure load is typically the most conservative of the different interpretative methodologies for estimating failure load from static load tests. Furthermore, it was developed for a very specific set of conditions and assumptions for driven piles that are not always appreciated in practice with the result that the method has been misused and applied in cases where it is inherently invalid (Kulhawy and Hirany 2009, NeSmith and Siegel 2009). But as with dynamic pile-driving formulas, the so-called Davisson Method has sadly endured in practice because of its relative simplicity-of-application compared to other methods, not because of its theoretical rigor or accuracy.

There are several additional issues involving static load tests that should be considered, any one of which can significantly affect the deduced failure load against which the calculated result from an analytical method is compared. To begin with, there are a number of procedural issues involving static load tests that are significant in terms of their potential effect on results:

- What is the source of the load, i.e. dead weight, reaction piles, or some combination?
- What is the load-application protocol, i.e. maintained load applied in intervals (and if so what duration, etc.) versus constant rate of penetration?
- How is the applied load measured, i.e. jack pressure, load cell, instrumentation within the pile?
- How are vertical displacements at the head of the pile measured?

The writer has had first-hand experience with some of these issues as discussed in detail in Horvath (2002). A condensed version is presented in the following section dealing with technical-needs fulfillment.

As noted previously in the discussion of liquefaction assessment, time can actually bring more chaos than clarification to a technical issue and that is certainly true of deep-foundation load testing. In addition to the above issues of defining the geotechnical failure load in a conventional static load test nowadays there is also the basic question of the type of load test, i.e. physical load application (which itself can be static or quasi-static (e.g. *STATNAMIC*)) or indirect based on dynamic field measurements and some wave-based technology such as the *Pile Driving Analyzer®* or *CAPWAP®* (discussed subsequently in the section on technical-needs fulfillment). Experience has shown that the same pile can have multiple interpreted geotechnical capacities depending on the methodology used.

The final issue is the time after installation at which the actual capacity is determined. It is now appreciated that the load-settlement behavior of driven piles is often temporally dependent in coarse-grain soil as it always is in fine-grain soil. In fact, some of the most useful and compelling published information in this regard has been based on observations at JFKIA by PANYNJ geotechnical engineers (York et al. 1994) as well as others working at JFKIA (Crincoli and Haider 2013). In most cases, and this has been the observation at JFKIA, the interpreted geotechnical ultimate resistance of piles increases with time which is referred to by most nowadays as setup although in the past some preferred the term soil-freeze.

For the sake of completeness, it is also worth mentioning pile-pile interaction effects in the form of group or cluster effects on pile resistance. It has been known for decades (Poulos and Davis 1980) that driving piles relatively close together so as to form groups or clusters typically results in individual piles capacities greater or less than that of a single pile driven in isolation, with the magnitude and sign of the difference depending primarily on soil type as well as other variables. This soil mechanics theory-based appreciation is a far cry from the multitude of older, now-deprecated geometry-based methods such as Feld's Rule that always resulted in lower group capacities (Chellis 1961).

The writer has observed that on occasion, specifically at JFKIA, the geotechnical ultimate resistance measured for a pile installed within a group was used as the ground-truth for assessing the accuracy of an analytical method. This practice should always be viewed in the proper context in that the measured pile resistance will reflect group interaction and net effects that are typically not accounted for in analytical methodologies which are typically based on single, isolated piles.

In summary, assessing the accuracy of any deep-foundation analysis methodology for geotechnical ultimate resistance in axial compression is not the straightforward process it has historically been, and often still is, taken to be. This is because quantifying a single-value 'true' geotechnical resistance for a deep-foundation element is at best subjective and at worst may be impossible. It is always necessary to clearly define how and when a measured ultimate resistance was determined. In fact, it may be better to indicate a range of ultimate resistances to simply and honestly reflect the uncertainty that is inherent in both the current state of practice as well as reality, i.e. there simply may not be a single-value ultimate resistance as Fellenius maintains. In any event, in the writer's publications (Horvath 2002, 2003a, 2003b, 2003c, 2004; Horvath and Trochalides 2004; Horvath et al. 2004a,b) related to the new interim analytical method for tapered piles that is under discussion here reasonable attempts were made to identify the basis for determining the ultimate resistance against which calculated capacity was compared.

Static Approach: Applications at JFKIA

A detailed explanation and numerical illustration of the writer's Modified Nordlund analytical model for tapered piles (but applicable to constant-perimeter piles as well) as well as an explanation of how the measured pile resistances were determined from static load tests were presented in Horvath (2002). Therefore this material will not be repeated here as the basic pile-resistance algorithm employed in the model has not changed.

However, what has changed, several times, since the original publication in 2002 is the site-characterization algorithm that seamlessly provides the necessary soil-property inputs to the pile-resistance algorithm. Therefore, it was of interest for this paper to re-analyze the two piles considered in the writer's 2002 study (a *Monotube*-brand tapered pile and a generic constant-perimeter steel pipe pile) using the current version (3.1) of the writer's site-characterization algorithm and compare these new calculated resistances to both the original ones as well as those measured in static load tests.

In addition, it is of interest to use the pile-resistance calculation module of the *CPeT-IT* software. Geotechnical resistances are based on the LCPC method that uses CPT q_c and f_s values directly and empirically. Unfortunately, this methodology has no obvious way to consider tapered piles so this software and method was used only for the constant-perimeter pipe pile.

Table 1 summarizes the results for the *Monotube*-brand tapered pile and Table 2 for the generic constant-perimeter pipe pile. The latter results require some elaboration as the toe of this pile (85 feet (26 m) BGS) was within a dense zone of the Pleistocene sand bearing stratum where soil consistency changes rapidly with depth as reflected in the CPT q_c values (Figure 13, CPT No. CTP-2). Thus even a relatively small difference in correlating the depths of the pile toe and CPT sounding could produce a significant change in estimated pile resistance for both the writer's and LCPC methods.

To illustrate this, it is worth noting that the *CPeT-IT* software calculates pile resistances for every 1 foot (305 mm) of theoretical pile penetration. Over a relatively short distance (a depth of 80 to 87 feet (24.4 to 26.5 m) BGS that brackets the actual toe depth) the calculated pile resistance using the LCPC method varied from 361 to 629 kips (1610 to 3000 kN), almost completely due to variations in calculated toe capacity. This is a very large range in values and illustrates yet another difficulty when comparing the accuracy of some calculation methodology to measured resistances.

Table 1. *Monotube* Pile - Comparison between Calculated and Measured Axial-Compressive Geotechnical Ultimate Resistance

Analytical method		Ultimate resistance, R_u, in kips (kN)	
		calculated	measured
writer	original study (2002)	452 (2010)	500 (2200)
	present study (2014)	464 (2060)	

<u>Pile Details</u>

1. *Monotube* Type 3N3J8x14
2. Embedded depth, D, = 64 feet (19.5 m) BGS
3. Driven close to CPT No. CTP-5

Table 2. Steel Pipe Pile - Comparison between Calculated and Measured Axial-Compressive Geotechnical Ultimate Resistance

Analytical method		Ultimate resistance, R_u, in kips (kN) calculated	measured
writer	original study (2002)	576 (2560)	450 (2000)
	present study (2014)	517 (2300)	
LCPC (*CPeT-IT*)		560 (2490)	

Pile Details
1. Outside diameter = 12.75 inches (323.9 mm)
2. Wall thickness = 0.5 inches (12.7 mm)
3. Closed toe
4. Embedded depth, D, = 85 feet (25.9 m) BGS

Seismic-Related Issues

There are two additional issues that significantly impact or otherwise relate to deep-foundation capacity at JFKIA. Both relate to the fact that for some years now the potential for a relatively significant earthquake occurring during the design life of structures built at JFKIA must be taken into account when designing most structures at the airport.

The first is by far the more significant and relates to the overall issue of liquefaction. Although there are varying and conflicting results from the different versions of the cyclic-stress method presented and discussed at some length earlier in this paper, there does appear to be a consensus that there is a reasonable liquefaction potential within the Pleistocene sand stratum (Upper Glacial Aquifer) that is the bearing stratum for all deep foundations at JFKIA. As a result, for some years now all deep foundations at JFKIA have a specified minimum tip elevation (presumably beyond the estimated zone of liquefaction) in addition to other installation criteria. In addition, the loss of some geotechnical capacity as a result of not only liquefaction but significant strength reduction as SF_L approaches unity should be considered in design.

However, the most direct and obvious impact of this need to consider liquefaction has been on the types of deep foundations considered acceptable for use at JFKIA. This is discussed in detail subsequently.

The second seismic-related issue involves the horizontal forces (lateral loads) imposed on pile caps when superstructure loads are resolved down to foundation level. Building and design codes typically place limits on the maximum magnitude of lateral displacement from lateral loads at the superstructure-foundation interface which, for piles, is typically the pile cap.

There are two key parameters that govern lateral displacement of a pile cap:

- the aggregate flexural stiffness of the piles in the group or cluster relative to the soil within a relatively shallow zone directly beneath the pile cap and

- fixity of the pile heads within the cap.

The former issue is relatively straightforward to deal with analytically and explains, in part at least, the trend to larger-diameter piles in recent years as flexural stiffness varies with diameter cubed so relatively modest increases in diameter can significantly increase flexural stiffness.

The latter issue has been receiving increasing interest in recent years although for reasons that are not explicitly related to JFKIA. Historically, this issue was dealt with trivially by assuming a perfect hinged condition so that no bending moment was, at least analytically, transmitted to the head of a pile. However, there is a price to pay for this analytical simplicity compared to assuming partial or full fixity and this is that, all other things being equal, lateral displacement is maximized when a hinged condition is assumed. The result of this has been situations where the number of piles in a group or cluster is governed by keeping lateral displacements within specified limits as opposed to being governed by vertical forces.

The writer is not aware of any specific instance at JFKIA where this situation has occurred although this does not mean it has not. It has certainly occurred elsewhere in the NYC metropolitan area but typically when relatively small-diameter, high-capacity micropiles socketed into bedrock are concerned. As a result, there is increasing interest in developing design-oriented methods for quantifying the degree of pile-head fixity as a function of embedment and connection details between piles and pile caps.

Deep-Foundation Product Technology

The evolution of deep-foundation alternatives at JFKIA, not only in terms of what has been used but what has been considered for use, is a rather remarkable microcosm of the evolution of both the technology used for deep foundations that derive all geotechnical support from embedment in coarse-grain soils as well as our understanding of how seismic issues can impact design. In later decades at least (i.e. from the early 1970s onward) this technological evolution tended to occur episodically (as opposed to more or less continuously) as a results of periodic building campaigns initiated by the PANYNJ. Because each of these campaigns required a significant number of deep-foundation elements, the PANYNJ Engineering Department used these opportunities to conduct an indicator-pile (test-pile) program early in the design phase of each campaign in order to take a fresh look at deep-foundation alternatives using both current as well as new/emerging technologies to develop the most cost-effective deep-foundation alternative for production use.

Horvath and Trochalides (2004) discuss three of these test-pile programs that were performed between the early 1970s and circa 2000. Some of this discussion as well as additional details will be provided in this paper. In addition, many of the specific pile types discussed in this section of the paper are described further as well as illustrated in Horvath (2003b).

To start at the beginning, information available to the writer suggests that timber piles (typically Southern Yellow Pine) with relatively modest (perhaps as low as 20 to 30 kips (89 to 134 kN) per pile in early years) axial-compressive maximum-allowable design loads were used predominantly, if not exclusively, in the first decades of construction at JFKIA. Timber piles were historically and to some extent still are a very popular, common, and cost-effective low-capacity friction-pile alternative in the NYC metropolitan area. With reference to Figure 11, timber piles would have been driven to achieve the requisite bearing, most likely based on the *Engineering News* dynamic formula which to this day is still a basic component of the NYC Building Code, within the uppermost portion (perhaps 10 feet (3 m) or less in some cases) of the Pleistocene sand stratum. Static load tests were likely rarely performed as the NYC Building Code historically waived load testing driven piles with maximum-allowable design loads that were 80 kips (356 kN) or less.

What is known with greater first-hand certainty by the writer is that by the early 1970s, i.e. roughly three decades after initial construction of the airport began, the de-facto 'standard' maximum-allowable design load of timber piles at JFKIA had increased to 60 kips (267 kN) per pile. Also by that time timber piles were always treated using creosote prior to installation.

During the 1972-1973 timeframe there was a test-pile program associated with the *IAB-STRAP* project that had as its centerpiece a proposed multi-level parking garage within the CTA. The focus of the pile-testing program was to investigate the feasibility of increasing the maximum-allowable design load of timber piles from 60 to 80 kips (267 to 356 kN) per pile. However, there were two other noteworthy aspects of this program, one of which would set the stage for a significant shift in later years.

The first was to install several types of steel pipe piles, each of which was driven closed-end and filled with PCC after installation, with a desired maximum-allowable design load of 120 kips (534 kN) per pile. These were:

- *Cobi Helcor* which was a constant-perimeter thin-shell pile that required a proprietary (Cobi) expandable mandrel for driving;

- *Raymond Step-Taper* which was a proprietary step-tapered thin-shell pile that required a proprietary (Raymond Pile Company) solid mandrel for driving; and

- *Monotube* Type Y partially-tapered thick-wall pile with a 0.95° taper angle.

For reasons that the writer does not recall, none of these pipe piles was ever subjected to conventional static load tests.

The second aspect of note for this early-1970s program involved two types of piles that were actively considered but ultimately were not pursued beyond the talking stage. Both involved the same concept of installing what amounted to a relatively large bulb of PCC just below the interface between the Holocene MTM and Pleistocene sand strata. It was anticipated that due to a combination of pile geometry and method of installation that very high (compared to timber piles) maximum-allowable design loads of up to 240 kips per pile (1068 kN) could be achieved. These were:

- the so-called *Franki* pile that was also being installed at the time by the Raymond Pile Company which called it a *Pressure-Injected Footing* (PIF) and

- the newly-developed *TPT*[85] pile that was essentially a precast version of the cast-in-place Franki pile/PIF.

The writer's recollection is that neither was pursued for the *IAB-STRAP* project due to concerns about crafting an entire project around a proprietary piling system that only one contractor would be qualified to bid on and install.

The next major building campaign was called *JFK 2000* and the deep-foundation testing associated with it was conducted during 1988 to 1990. This program was the source of both the in-situ (SPT and CPT) testing as well as piles used in this paper as well as several earlier publications by the writer. This testing program was much more comprehensive than the one conducted almost 20 years earlier and included timber, *Monotube* Type J (ω = 0.57°), and generic constant-diameter, thick-wall steel pipe piles of various diameters that were driven closed-end. This also marked the first time that drilled shafts (both

[85] This acronym stood for 'tapered pile tip'.

conventional and post-grouted) were tested at JFKIA. The latter in particular was quite innovative for its time in the NYC metropolitan area. There was also a marked increase in the desired maximum-allowable capacity per pile. For example, capacities in the range of 200 to 240 kips (890 to 1068 kN) were sought for the *Monotube*-brand tapered piles.

It is the writer's understanding that the *Monotube* pile emerged from this testing program as the most cost-effective choice although at least one subsequent project is known to have used generic constant-diameter, closed-end steel pipe piles. Not surprisingly, in the writer's opinion, generic drilled shafts did not prove to be cost-competitive although post-grouted shafts apparently did. However, this alternative apparently presented potential sole-source/proprietary contractual hurdles at the time so their use was apparently not pursued further.

As noted previously, at some point toward the end of the 20th century the PANYNJ apparently made a policy decision that the direct (shaking) and indirect (liquefaction) effects of earthquakes had to be considered in the design of most new structures at JFKIA. The timing and extent of this decision is not known to the writer at the present time. In any event, this decision had a profound effect on the directions subsequently taken in deep-foundation product technology at JFKIA. Specifically and as a result of liquefaction considerations, for the first time at JFKIA that any deep-foundation element would have to be installed to some minimum toe elevation. This meant that, overall, deep foundations would have to be substantially longer than used historically. It is interesting to reflect on the fact that shallow, high-capacity piles such as the Franki/PIF and TPT would have been banned outright from further use had that path been taken in the early 1970s.

Further, as a result of lateral-load demands at the head of deep foundations this made larger-diameter (at least at the head) piles more attractive to use. Such piles would inherently have greater axial-compressive capacity as well.

Thus the net result of seismic design requirements pushed deep-foundation technology at JFKIA toward using longer, larger-diameter/-capacity elements by the end of the 20th century. This set the stage for the third and final building campaign addressed in this paper that occurred about a decade after the preceding one, circa 1998-2000.

At the cusp of the new millennium there was substantial passenger-terminal construction within the CTA as well as construction of the elevated AirTrain system to connect the CTA passenger terminals with remote parking lots and off-site mass-transit stations. A significant aspect of this work was that there does not appear to have been a formal design-phase deep-foundation testing program initiated by the PANYNJ as with the two building campaigns discussed previously. Rather, it appears that contractor initiative was the primary motivation behind the technology advancement that ultimately occurred during these years.

It appears that by the end of the 20th century deep-foundation technology at JFKIA was defined by *Monotube* Type Y ($\omega = 0.95°$) partially-tapered piles with an allowable axial-compressive capacity of 300 kips (1335 kN) or about an order-of-magnitude greater than the earliest timber piles used there. This is not surprising in view of seismic-design considerations pushing pile capacities higher. However, piles of such capacity were apparently at the limit of *Monotube*-pile technology which uses cold-formed steel sections. This manufacturing process places well-defined limits on the wall-thickness of the sections that can be created. Thus the maximum axial-compressive capacity of a *Monotube* pile is actually limited by structural, not geotechnical, considerations, specifically driving stresses during installation.

The desire to push the edge of the pile-capacity envelope beyond the capabilities of the *Montotube*-brand tapered steel pile led to the development of the *Tapertube*-brand tapered steel pile (Horvath et al. 2004a, 2004b). The *Tapertube* pile mimics the geometry of the *Monotube* but is constructed of hot-rolled steel sheet that allows for piles with significantly greater wall thicknesses compared to the *Monotube*. Thus the structural limitations that are inherent in *Monotube* piles do not exist with *Tapertube* piles. Allowable axial-compressive capacity is limited strictly by geotechnical considerations.

The *Tapertube* pile resulted in maximum-allowable axial-compressive capacities of up to 400 kips (1780 kN) per pile to be achieved at JFKIA and environs on a regular basis soon after commercial production of what the writer terms the Type II design ($\omega = 0.95°$). The writer is aware of cases where ultimate geotechnical resistances in excess of approximately 1000 kips (4500 kN) per pile were demonstrated by static load test.

Before concluding this discussion of the evolution of deep-foundation product technology at JFKIA, there is one additional item of note that relates to this topic. This is the use of hot-rolled, spiral-wound, constant-diameter steel pipe for the extension portions of tapered steel pipe piles[86]. This use of spiral-wound pipe is noteworthy as for many years the PANYNJ allowed only seamless pipe to be used in any pipe-pile application. The concern was, apparently, that spiral-wound pipe could suffer localized weld failure during hard driving that would permanently compromise the integrity of the pile.

Summary of Key Points

It is very difficult to efficiently summarize all the geotechnically-related technological changes that have occurred in the 70-plus years since construction of JFKIA first began and how they impacted the way that foundation engineering designs are crafted to meet the needs of a large commercial international airport. However, in the writer's opinion the evolution of the concept of plate tectonics and the concomitant recognition of the NYC regional seismic potential and all that it entails in terms of liquefaction and seismic loading stand out as having the single largest influence on foundation design. Even if geotechnology were still limited to using sliderules; drilling borings with SPT samples; and relying solely on static load tests to determine pile resistances the need to install piles to greater depths would still have evolved because of changes in the understanding of geology. Furthermore, it is not unlikely that those piles would still have been *Monotube* (they existed since the 1920s) and, possibly, later on *Tapertube* piles as well.

Because liquefaction in particular has influenced how foundation needs at JFKIA have been and are currently satisfied, it is logical that future attention should be focused on this issue to the greatest extent practicable. This means that, among other things, greater use should be made of the CPTu in general and sCPTu in particular during routine site investigations. Investigation of the general issue of soil-aging and the possibility of liquefaction in the past should also be pursued as these influence the assessment of the potential for liquefaction in the future.

With regard to technological changes that have not progressed or developed to the extent that they could or should have, the writer posits that it is surprising that greater efforts have not been expended by both academic researchers and industry stakeholders to advance static-approach capacity-analysis methods as well as the wave equation for tapered piles. The goal should be to develop analytical methodologies that are both reasonably accurate and straightforward to use in routine practice.

[86] Generic hot-rolled, constant-diameter, thick-wall steel pipe has always been used as the constant-diameter portion of *Tapertube* piles. In recent years it has also been used as the constant-diameter portion of *Monotube* piles as an alternative to the proprietary Type N cold-formed extensions.

TECHNICAL NEEDS: FULFILLMENT

Introduction

This third and final primary section of the paper deals with how the foundation designs developed by design professionals have not only been implemented by contractors but how design professionals have conducted CQA to verify contractor work for compliance with contract documents (plans and specifications). As with the preceding section on needs assessment by design professionals, this discussion focuses on how deep-foundation installation and concomitant integrity and capacity verification has changed over the 70-plus years.

Pile-Driving Equipment

The writer has no specific knowledge about the nature of pile-driving equipment used at JFKIA up until circa 1970. However, based on general practices for timber piles in the NYC metropolitan area it is likely that single-acting steam/air hammers were used, initially with steam as the propulsion fluid and later compressed air.

It is known for certain that at the time of the 1972-1973 *IAB-STRAP* test-pile program that single-acting hammers driven by compressed air were the de-facto standard for pile installation at JFKIA. This testing program included a trial of a double-acting air hammer because the rated speed (blow rate) was approximately twice that of typical single-acting air hammers in use at the time. The thought was that if double-acting air hammers (and, by extension of the logic, differential air hammers as well) performed as well as single-acting air hammers that the double-acting and differential hammers would likely be the hammers-of-choice because of their potential for greater productivity.

However, use of the then-new-and-novel dynamic measurements (discussed subsequently) showed clearly that the double-acting hammer had drastically lower/poorer driving efficiency compared to the single-acting hammers used at the same time, even though a representative of the manufacturer of the double-acting hammer was on-site and pronounced their hammer to be in normal working order. As a result of this testing program, double-acting and differential hammers were, at least for some time afterward, banned from use on PANYNJ projects.

To the best of the writer's knowledge, neither diesel nor vibratory hammers were ever used to any significant extent, if at all, for permanent foundation piling on projects at JFKIA. In general, diesel hammers were never as popular in the NYC metropolitan area as they have been in many other parts of the U.S. and world. This is likely due to the extreme simplicity and robustness of single-acting air hammers that pile-driving contractors in the NYC metropolitan area have long possessed and amortized. However, in recent decades the hydraulic hammer has emerged as the hammer-of-choice on projects at JFKIA because of its superior operating characteristics in terms of variable rated energy and operational speed. However, the single-acting air hammer still remains viable for use, at least in principle and concept, and it is possible that on any given project at JFKIA such a hammer could still see use.

Pile-Capacity Verification

Overview

Historically, both before and after the emergence of modern foundation engineering based on soil mechanics principles and to the present it has been routine to perform load tests, i.e. field-verify the axial force-displacement behavior of individual deep-foundation elements, on all but the smallest of projects. This is completely opposite the state-of-practice for shallow foundations of all types where, with few exceptions, designs are based solely on theoretical calculations and performance-monitoring of built foundations to verify these calculations rarely conducted.

Much more recently, lateral-load testing of deep foundations has become more common in routine projects because of the need for seismic design in areas such as the NYC metropolitan area that were traditionally assume to be aseismic. This trend has affected JFKIA.

Because of the significant role that post-construction axial-capacity verification in particular plays with deep foundations of all types, it is not surprising that there has been enormous technological change in the 70-plus years that JFKIA has been in existence. In particular, over the last 40-plus years there has been the development and evolution of a testing alternative based on pile dynamics that both complements and, in some cases, replaces the traditional static approach. As it turns out, JFKIA was actually one of the earliest, if not the earliest, uses of this alternative dynamic testing in the NYC metropolitan area.

Static Methods

In-situ testing of the geotechnical axial resistance of deep foundations using static loading is intuitive and so had its origins well before the emergence of modern soil mechanics in the first half of the 20th century. Perhaps as a consequence of the intuitive 'we have always done it this way' nature of static load testing there is a tendency to trivialize the test protocols and assume that such tests always produce a single-value result, i.e. 'the correct answer' or ground-truth.

As discussed in some detail earlier in this paper and in detail in Horvath (2002), reality is quite the opposite. There are many variables involved in the traditional static load test and its interpretation. These involve:

- The hardware and associated protocol of the test setup and how the applied load is generated (i.e. dead weight, reaction piles or ground anchors). The primary concern here is how the load-generation system influences the stress-state in the ground and, therefore, the measured geotechnical resistance of the tested deep-foundation element.

- The hardware and associated protocol of how the applied load is measured (i.e. jack pressure, external load cell, internal force measurements). The primary concern here is accurate measurement of the forces actually applied to the head of the deep-foundation element.

- The hardware and associated protocol of how vertical displacements of the pile head are measured, i.e. dial gauges, tensioned wires with scales and mirrors, high-precision optical surveys. The primary concern here is that the recorded vertical displacements of the pile head are total (absolute) displacements relative to a non-displacing datum and not inadvertently and incorrectly relative displacements due to a reference frame that is not spatially fixed. Displacements in the latter case will typically be smaller than in the former and thus present an overly optimistic view of the foundation-element's behavior.

- The analytical methodology used to determine the 'failure load' when the goal of the test is to cause geotechnical failure as opposed to simply proof-test to some predetermined load level, e.g. twice the design capacity (double design). This issue has been discussed at some length already and relates to the fact that the overall load-settlement curve does not always exhibit a plunging type of behavior that would make failure-load determination reasonably straightforward.

The net result of all these factors is that at best the failure load produced by any static load test is, in reality, a range of loads. At worst, there is no true failure load as has been suggested by Fellenius and a static load test simply produces a non-linear force-displacement curve that can then be used as part of a displacement-based analysis or design process.

The writer's first-hand experience at JFKIA is that the PANYNJ was ahead of the overall profession in recognizing the importance of most of the issues itemized above. For example, as early as the early 1970s they recognized the importance of using electronic load cells and not relying on jack-pressure measurements to determine applied forces. In addition and ever more noteworthy in the writer's opinion was their use of high-precision optical surveys that were referenced to special deep benchmarks installed specifically for the load tests as the primary mode for measuring vertical displacements of the pile head. Whether this practice has continued in recent years is not known nor are the procedures used when load tests are performed under the direction of consultants to private owners.

Also unknown to the writer at the present time is whether modern quasi-static load-test methodologies such as the STAT*NAMIC* test have ever been used at JFKIA. The extent to which lateral load tests have been performed to measure the force-displacement relationship at the heads of piles is also not known the writer at the present time.

Dynamic Methods

Formula-Based

Pile-driving formulas have long been a staple of CQA and contractor compliance for driven piles. Despite the fact that these formulas have long since been shown to be technically flawed they remain legal in some codes (including the NYC Building Code) and thus in use, especially on smaller projects, because of their extreme simplicity and ease-of-use.

Up until the 1972-1973 *IAB-STRAP* pile-testing program such formulas would almost certainly have been the norm for use at JFKIA. However, by the time of that program the PANYNJ was aware of the fact that these formulas left something to be desired in terms of how well capacities calculated with these formulas correlated with static load-test results. Therefore, this testing program included two distinct components related to capacity verification:

- an effort to empirically modify the constant coefficients of the *Engineering News* formula to provide better correlation with actual capacities measured in static load tests and

- a first-time use of a then-new wave-based technology.

As it turned out, the latter revolutionized the installation CQA of not only driven piles but all types of deep foundations and is discussed in the following section.

Wave-Based

A strong case can be made that the single most significant technological advancement related to all types of deep foundations that has occurred in the last several decades has been the development and evolution of 1-D stress-wave dynamics as an analytical tool. Simply stated, the ability to measure various physical parameters as nominally 1-D stress waves travel through a deep-foundation element either during initial installation (in the case of driven piles) or after installation (in the case of both driven piles as well as various drilled-in elements) and then numerically process the measured data to make various assessments involving geotechnical capacity and/or compositional integrity has revolutionized all aspects of deep-foundation design and construction.

The temporally-longest and most widespread use of stress-wave dynamics has been with driven piling. It is of interest to note that it was the initiative of the PANYNJ that led to one of the earliest, if not the earliest, uses of dynamic measurements in the NYC metropolitan area at JFKIA in 1972 as part of the *IAB-STRAP* test-pile program. In that initial usage it was actually pile-hammer efficiency that was of significant interest (the measured low efficiency of double-acting hammers is what led to their not being allowed on future PANYNJ projects) although there was certainly interest in the estimated geotechnical capacities as well.

Subsequent to this initial usage, the PANYNJ began to routinely use of dynamic measurements on projects of all size at JFKIA and it is the writer's understanding that this continues to the present. Although the basic concepts have not changed the field-measurement hardware as well as the analytical software has gone through many stages of improvement in the 40-plus years since the technology was first used at JFKIA.

It is the writer's perception that wave-based dynamics have evolved to be the CQA compliance tool of choice on many driven-pile projects and JFKIA is no different in this regard. While this is an improvement over the long-discredited pile-driving formulas there is a downside nonetheless. This is because from what the writer has seen of wave-based dynamics (which dates back to first-hand exposure to the 1972 use at JFKIA) they routinely leave something to be desired in terms of capacity-estimation when tapered piles are involved.

The reason for this is not hard to understand. To date, all wave-based dynamics, whether used for the office-based wave-equation software discussed earlier in this paper or the field-based methods under discussion here, are based on the same model of 1-D wave propagation through a cylindrical rod that has ground resistance only from the two classical mechanisms of shaft friction and toe bearing. Therefore, as the 1-D wave travels through the rod there can only be sliding resistance along the shaft interface with the surrounding ground and compressive resistance of the ground underlying the toe. The true physical behavior of a tapered pile, i.e. cylindrical cavity expansion, is not even remotely captured by this model. As a result, it should come as no surprise that a 1-D wave-based analytical methodology as currently formulated does not provide a good match with actual behavior for tapered piles.

Summary of Key Points

While the basic concept of installing piles by impact driving has not changed throughout the course of human history, the many theoretical and technological developments based on and around 1-D stress-wave mechanics have certainly clarified the understanding of the pile-driving process as well as allowed for various predictive methodologies to be developed. In particular, methods based on field measurements at the time of driving or post-driving restrike of piles have gained universal acceptance as a CQA capacity-verification tool that reduces, if not eliminates in some cases, the need for traditional static load tests.

SYNTHESIS AND COMMENTARY

Introduction

The foundation-related experiences at JFKIA are of greatest interest and practical use to civil engineers for the broader insight they provide into various aspects of foundation analysis, design, and construction that can be used to varying extents on other projects worldwide. This section of the paper provides the writer's interpretations and opinions of the broad insights of practical relevance that can be drawn from them JFKIA foundation experiences that have been presented in this paper.

Site History

As this paper has clearly demonstrated, even a relatively remote site with natural site conditions that are a deterrent to human development can have a surprisingly rich and varied history of human activity. In the specific case of JFKIA, this prior history has taken the form of prior filling with a wide variety of soil-particle sizes, installation of thousands of driven piles, installation of permanent well casings, and remnants of structure and road construction. Any and all of these features have the potential to obstruct the installation of deep foundations for future construction.

Nowadays, the Web makes researching old publications, maps and newspapers in particular, relatively easy so there is no excuse not to perform this exercise for even the smallest of projects. As can be seen from the numerous citations in this paper, such material can be a rich resource for learning about prior site history that has direct impact and, therefore, relevance to foundation design and construction.

Geology

Geology has always been an important civil engineering tool but never more so than at present. This is because of the now-appreciated phenomenon of plate tectonics and the consequences it has at any site where foundations are to be constructed in terms of defining the basic subsurface conditions, past seismic activity, and future seismic potential. In light of this, it is the writer's strong opinion that anyone engaged in foundation design and construction should have a basic education in geology and even the smallest of projects should include a basic assessment of site geology.

Unfortunately, it is the writer's first-hand experience that the current trend in civil engineering education, in the U.S. at least, has been to reduce student exposure to geology in the mandatory curriculum. Thus a broad recommendation is that it should be a priority to reverse this trend.

A more-specific suggestion is that it would seem useful to make greater use of paleoseismology studies in general and paleoliquefaction studies in particular where appropriate subsurface conditions and seismic history/potential exist. Such studies can provide additional data points for constructing the seismic history of any area which enhances the ability to rationally estimate future seismic potential for an area.

Paleoseismology and paleoliquefaction can also be useful tools when trying to estimate the behavioral age of a soil stratum. As noted in the extensive discussion of liquefaction potential at JFKIA that was presented in this paper, in recent years the importance of behavioral age when interpreting SPT, CPT, and V_s data to estimate either future liquefaction potential or a lower-body earthquake magnitude and/or ground-surface acceleration that triggered liquefaction in the past is now appreciated.

Site Characterization

The advances in site characterization over the past 70-plus years have been nothing short of phenomenal. Not only have numerous in-situ testing devices been developed but there has been considerable research into developing empirical correlations for myriad soil properties using measured data produced by these devices. Even the relatively crude SPT, which would have been one of the few subsurface-investigation tools available to civil engineers in the U.S. in the 1940s, has benefitted substantially from research related to in-situ testing.

It is significant to note that some of the soil properties that can be estimated from in-situ tests, especially those involving stress state in coarse-grain soil, cannot be estimated in routine practice using the conventional methodology of soil sampling followed by laboratory testing. As discussed earlier in this paper, such stress-state properties, especially K_o, are particularly relevant to deep-foundation analysis and design as it has been shown conclusively that shaft friction for all types of deep foundations in soil correlates well with the K_h/K_o ratio.

Unfortunately, as noted by Mayne (2012) and DeGroot (2013) and consistent with the writer's personal experience the enormous advances in site characterization have not made their way into either undergraduate education or routine practice to the extent they could or should. Even worse, there actually seems to be a regression in terms of the overall quality of site characterization in routine practice. For example, the writer has noted in recent years that even something as simple as noting the type of hammer-drive system used when performing the SPT is not included on most boring logs. The utility of making this most basic of observations, which can be used as a first-order estimate of driving efficiency, was recognized decades ago so should long ago have made its way into routine practice.

With this current state of practice in mind, the following suggestions are made from the perspective of reversing this trend for the overall benefit of the state-of-practice:

- There is no reason why a wide range of soil properties should not be estimated routinely on even small projects, especially when coarse-grain soils are involved. Even if only SPT N-values are available there are empirical correlations that produce pseudo-q_c values that allow CPT-based soil-property algorithms and empirical relationships to be used as has been illustrated in this paper.

- There should be greater use of both CPTu and sCPTu soundings to both complement and replace conventional borings in all types of soil conditions. There is now a substantial suite of empirical correlations for both coarse- and fine-grain soil that allows reliable estimation of a wide range of soil properties of use in geotechnical and foundation analyses.

- There should be a move toward integrating site characterization with geotechnical and foundation analyses to make maximum use of modern site-characterization and computational capabilities to breathe new life and enhanced accuracy into well-established analytical procedures.

Deep-Foundation Capacity Determination

Axial-Compressive

Overview

In the writer's opinion, the construction of deep foundations for which axial-compressive loads are the governing factor should, nowadays, always involve a combination of pre-construction design complemented by field verification during construction. On major projects, an intermediate step of a field-test program as part of the overall design process for the purposes of deciding on the most cost-effective deep-foundation alternative may also appropriate. When done properly, this combination of design- and construction-phase activities should have a synergistic outcome.

The design phase should include static-capacity calculations as its basic component, especially when liquefaction is an issue as this is the only way to rationally account for post-installation reduced capacity of deep-foundation elements within fully- or nearly-liquefied zones. The design phase should also include drivability assessments using wave-equation software whenever driven piles are going to be used unless there is already site-specific experience that indicates there are no drivability issues and potential pile-driving contractors already know the most-efficient hammer to use based on prior experience.

The nature of construction-phase field verification for CQA purposes depends significantly on the specific type of deep-foundation elements involved. In general, both dynamic- and static-based methodologies need to be used.

The most significant aspect of this overall process is the fact that how design- and construction-phase activities are blended together to produce a final foundation product has been very fluid and ever-changing over time as a result of technology development and evolution. When construction of JFKIA first began in the 1940s and for some decades thereafter, the ability to perform a meaningful design beforehand using analytical methodologies was largely non-existent. This placed the entire burden on field verification during construction.

This has changed over time, of course, and at present with the availability of various drilled-in technologies there is even greater diversity as different types of deep foundations have followed different design paths due, primarily, to differences in installation. Added to this is the latitude of professional judgment which is especially broad on projects in the private sector.

In the following several sections the writer's observations and opinions concerning the various design and construction-verification tools available for deep foundations are presented. In keeping with the scope of this paper, the focus is on driven piles.

Dynamic (Formula-Based)

In the writer's opinion, it is unfortunate that building codes (New York City being one) still allow use of the long-ago-deprecated pile-driving formulas as the primary, and in some cases sole, CQA capacity-verification tool during construction. Even more unfortunate is the fact that some design professionals continue to devote resources to tweak these formulas and even adapt them to current LRFD-based design methodologies (Allen 2005). The simple fact of the matter is that all of these formulas are based on a fatally-flawed physical model for pile driving. Therefore, no matter how simple these formulas may be to use that should not be an argument for their continued use in practice. Just because it is easy to do something the wrong way is no defense or justification for doing something the wrong way.

Dynamic (Wave-Based)

As has been known for well over 50 years now, the correct physical model for traditional impact-driving of piles is, in general, that of a 1-D stress wave traveling through a relatively slender rod. This model has produced both analytical software (wave equation) used in drivability assessments as well as various capacity-verification methodologies for field use during construction. However, as noted previously both of these tools suffer from the fact that their traditional formulations do not properly capture what happens during the driving of tapered piles. Surprisingly, in the writer's opinion, there does not appear to have been any interest or effort to modify this wave model for tapered piles.

Both the wave-equation software and wave-based field methods are very useful, practical tools that the writer has used personally since the earliest years of these methodologies' existence more than 40 years ago. However, the writer has perceived a trend in recent years that at times there can be an over-reliance on wave-based methods, especially those employed in the field for CQA purposes during construction. The notion that there is 'a load test in every hammer blow' is seductive to even experienced design professionals and can lead to reduced use of the other analytical tools discussed previously that are also important components of successful installation of a pile-foundation system. The reality is that the wave-based field methods have their limitations, especially when tapered piles are concerned, and do not always yield perfect agreement with static load tests even when restrike, as opposed to end-of-initial-driving (EOID), results are used.

Static (Calculations)

Static-calculation methods for estimating the axial geotechnical resistance of driven piles were the first improvement to deep-foundation practice brought on by the development of modern soil mechanics as they could be used even before the development and widespread availability of digital computers that were necessary for wave-based technologies. While static-capacity calculation methods continue to be improved for some types of deep foundations such as drilled shafts, it appears that similar efforts have not been maintained for driven piles in recent years, at least not to the extent they could or should.

This is quite unfortunate in the writer's opinion as there is great potential for incorporating some of the more-significant developments of recent decades (e.g. Kulhawy's observation of the importance of pre-installation K_o through the K_h/K_o ratio and Kodikara's work related to the third capacity mechanism of cylindrical cavity expansion for tapered piles) by integrating site characterization and capacity analysis into a single, seamless process. The LCPC capacity method that is integrated into the CPT software *CPeT-IT* that was used in this paper is an example of what can be achieved.

Another issue that would appear to be important when performing static-capacity calculations is to consider reduced soil resistance within zones where $1 < SF_L \lesssim 1.5$. Although dealing with fully-liquefied zones, i.e. $SF_L < 1$, is obvious there appears to be less appreciation of the fact that significant excess pore pressures and concomitant significant reductions in soil shear strength can occur as SF_L approaches a value of 1.

Static (Load Testing)

Given the relatively high cost of static load testing and thus the relatively significant financial investment that such tests represent on a project, the writer has long been surprised that greater thought and attention-to-detail were not routinely put into conducting and interpreting them. Some specific areas to which, in the writer's first-hand experience, particular attention should be paid include:

- All components of the test should be shielded from sunlight using tarps or similar temporary protection. Exposed load-test components make for great public-relations photographs but compromise various aspects of test integrity due to thermal expansion and contraction of various test components as a result of direct exposure to sunlight.

- Jack pressure should never be used as a way to measure applied forces as it is well-known to be consistently incorrect due to piston friction. As a minimum, a calibrated electronic load cell should be used. This is crucial as jack pressure-based measurements tend to overestimate forces which always produces an overly optimistic projection of deep-foundation capacity.

- There is simply no substitute for high-precision optical surveys referenced to a fixed benchmark established a considerable distance (tens of feet) away from the test as the primary way to measure displacement of the head of the deep-foundation element. In many cases the common use of reference beams combined with dial gauges, wires, etc. places the beam supports too close to both the deep-foundation element being tested as well as the supports for the transfer (reaction) beam used as part of the loading mechanism. Again, this is a critical issue as conventional measurement methodologies using reference beams tend to underestimate vertical displacements of the head of the deep-foundation element which again always results in an overly optimistic estimate of the force-displacement behavior of the element.

- When it comes to bad habits still in use with driven piles, second only to the continued use of the so-called dynamic formulas is the near-exclusive use of Davisson's method for interpreting failure load in static load tests. This method has been applied routinely over the years without fully understanding its limitations and the fact that it typically produces an estimated geotechnical failure load that is much more conservative than other methodologies.

With regard to the final item, the real issue in some cases is whether or not there is even an unambiguous single-value geotechnical failure load for a deep-foundation element. As discussed earlier in this paper, Fellenius has championed the positions that:

- the bearing-capacity concept and mechanism does not apply to the toe-capacity component of deep foundations in general and

- there is no clearly-defined failure load in many cases for deep foundations embedded completely in soil.

Whether or not one agrees with the former the latter is certainly true. This means that deep foundations in soil should be designed on the basis of allowable settlements, not strength. This is certainly the case when there are significant drag loads to contend with as Fellenius (2014) has illustrated well.

Lateral

With the requirement to design new structures for seismic loads, the lateral-load behavior of deep foundations has taken on a whole new importance throughout the NYC metropolitan area as it is now an issue that must be addressed on virtually every deep-foundation project. On the one hand, the behavior of individual deep-foundation elements subjected to lateral and/or moment loading is reasonably well understood and estimated for design purposes using computer software based on the *p-y* curve method. However, most deep-foundation elements, including driven piles, are typically used in groups or clusters connected with a pile cap so it is the performance of the overall system of pile cap plus multiple deep-foundation elements that is most relevant in practice.

Unlike axial capacity, which is typically designed based on strength, the design for lateral loading is typically based on a maximum allowable value of lateral displacement of the cap. As a result, and somewhat unexpectedly, the key issue that has emerged in recent years in practice is that of fixity between the head of the deep-foundation element and the pile cap in which it is embedded.

Historically, this connection was dealt with trivially at best and simply assumed to be an ideal hinge, i.e. unrestrained rotation was assumed for the head of the deep-foundation element that was embedded some nominal depth into the cap. However, all other things being equal the magnitude of lateral displacement is noticeably less if there is some fixity (i.e. restraint against rotation) of the head of the deep-foundation element, with minimum displacement occurring for the idealized limiting case of full fixity (no rotation).

In recent years, it has been found that in some cases, usually when relatively small diameter/high-axial-capacity elements are involved which happens most often with micropiles socketed into bedrock, that the assumption as to the degree of fixity between pile and cap can actually impact the overall cap design to the point of governing the number of piles required. As a result, the issue of fixity is no longer trivial in practice.

There are, at present, some empirical rules-of-thumb as to when head fixity of a deep-foundation element within a pile cap can be assumed. However, the science, if any, on which these rules were developed is not clear. Therefore, it would seem to be a priority to conduct meaningful, technically-sound research into developing a rational analytical methodology for defining the degree of fixity between the head of a deep-foundation element and pile cap.

In the writer's opinion, this does not appear to be a simple issue to deal with as not only is the relative rotation between the head of a deep-foundation element and cap an issue but also the relative rotation between the cap and the structural element (e.g. building column) that it is supporting. Thus in reality this is a relatively complex soil-structure interaction problem that is project-specific in nature. As a result, it is not immediately obvious to the writer how this can be reduced to a simple rule-of-thumb to be used in routine practice.

A separate issue that does not appear to have been fully appreciated in practice is to consider the effect of a reduction in soil strength on lateral-load behavior. As is well known, it is always the ground directly below the pile cap that controls lateral-load behavior. This is true even if this ground is 'poor' and ignored for axial-resistance purposes as is the case at JFKIA. Thus ignoring the Holocene hydraulic-fill and MTM strata for axial-compressive resistance does not mean that these strata can be ignored for lateral-load behavior.

The point being made here is that if full or even partial liquefaction occurs within the saturated portion of the Holocene hydraulic-fill stratum, a distinct possibility as shown in Figure 28, then there will be a reduction of soil resistance to lateral loading within the zone of soil that is most affected by and controls lateral loading. Therefore, liquefaction assessments are important throughout the entire soil stratum, even within zones that may not contribute to axial resistance of a deep-foundation element.

Further to this point, the benefit of performing some type of ground modification/improvement to relatively shallow soils that dominate lateral-load behavior should also be considered. Even though the window of opportunity for performing deep ground modification may have passed for a given site, as it did decades ago at JFKIA, that does not mean that shallow ground modification is not technically viable in the present for the purposes of improving lateral-load behavior.

FUTURE TRENDS

Forecasting future trends and developments in technology is an exercise that is inherently fraught with uncertainty as projections are made based on what is currently known. As a result, such forecasts typically envisage evolutionary tweaks and improvements to the status quo as opposed to some revolutionary insight. This is understandable as it is impossible to know in the present what currently-unknown material, product, concept, event, situation, etc. may develop and evolve in the future that might significantly impact technology and take it in a direction completely unknown in the present. Nevertheless, it is useful to engage in such prognostication based on observing those trends already known as it at least provides some talking points which is better than having nothing at all.

To begin with, there is the global issue of how JFKIA will be affected by the ongoing rise in sea level, both directly and indirectly. There is no indication that this rise in sea level will abate or even reverse itself anytime soon. If anything, it may occur at an increasing rate. The direct impact is, of course, the potential for increasing flooding potential of the airport property unless efforts are made to create a system of levees or dikes similar to what already exists at LaGuardia Airport.

An indirect impact of continued sea-level rise that would be impossible to control in any reasonable way is that there will likely be long-term increases in the static porewater pressures, u_o, within the Upper Glacial Aquifer that is the bearing stratum for all deep foundations at JFKIA. Such an increase would lead to a decrease in effective stresses within that stratum. This could, in principle, lead to permanent reductions in axial-compressive geotechnical resistance under static loads for all installed deep foundations as well as increased liquefaction potential.

A more-specific trend is that there will likely be increasing emphasis on evaluating the carbon 'footprint' or impact for both construction activities as well as the long-term performance of structures. The latter typically is significantly impacted by energy usage for lighting and temperature control (heating and cooling) of enclosed space. This may mean that there is greater use of energy piles, i.e. civil-mechanical engineering hybrids that can both support a structure as well as function as part of a heat-exchange system between structure and ground for purposes of heating and/or cooling. To date, energy piles are typically drilled shafts, a.k.a. drilled or bored piles, as the necessary mechanical tubing is attached to the rebar cage for the shaft prior to installation. If this trend persists this could mean a significant, permanent paradigm shift for the type of deep foundations used for future construction at JFKIA. On the other hand, it is possible that some new, innovative type of driven pile is developed that has all the necessary mechanical hardware pre-installed in the pile prior to driving.

Note that the use of energy pile could conceivably be extended to transportation structures such as bridge abutments and piers as well. This is because there is the potential to use heat extracted from the ground to prevent the seasonal icing of bridge decks.

Finally, there will likely be incremental improvements in technologies already in existence, especially for the wave-based dynamic measurements performed during construction for CQA capacity verification. There is already hardware available to allow such measurements to be made without having a trained engineer or technician on-site. Hardware can be installed by site personnel and the data sent via a cellular-telephone connection to any off-site location in the world for review and interpretation. This has the potential to make such measurements even more cost-effective which could increase their use and the concomitant reliance on the outcomes from these measurements.

ACKNOWLEDGEMENTS

To a significant extent, this paper reflects and draws on the sum total of the writer's overall 42-plus year professional engineering career that includes decades of general experience researching and working with site characterization and tapered piles plus specific experience at JFKIA. However, this paper would not have been nearly as complete without the generous input and sharing of knowledge from two individuals in particular.

First and foremost is Donald L. 'Don' York, P.E. who was the writer's initial 'boss' during employment at the PANYNJ in the early 1970s. In later years Don generously provided the writer with information related to the *JFK 2000* test-pile program conducted by the PANYNJ in the late 1980s.

Second is Stanley 'Stan' Merjan, P.E. whom the writer met through Don York early in the writer's PANYNJ employment. At the time, Stan was the leading technical figure at then-independent Underpinning & Foundations (U&F, now a Skanska subsidiary). U&F was then and still is a major deep-foundation contractor in the NYC metropolitan area. Stan was the prime mover behind development of the TPT 'bulb' pile in the early 1970s and then reprised this innovative development by having a hand in development of the *Tapertube* tapered steel pipe pile roughly 40 years later.

The writer is also grateful to U&F Skanska for making available data related to the development and use of the *Tapertube* pile at JFKIA circa 2000.

However, opinions expressed in this paper are those of the writer and do not necessarily reflect those of any other person, business, organization, or institution.

LIMITATIONS

The use of person/organization/business names, trade names, and trademarks in this paper is solely for identification and other factual-documentation purposes deemed necessary for the historical, informational, and educational goals of this paper. No judgment, endorsement, or recommendation of any named person, organization, business entity, material, or product is expressed or implied by the writer.

APPENDIX

Updated Site-Characterization Algorithm for Coarse-Grain Soil

Algorithm Background and History

As a personal and professional courtesy of the senior author, the writer was provided with an original paper copy[87] of the comprehensive research report on soil properties produced by Kulhawy and Mayne (1990) soon after its publication. In the writer's opinion, this report was a seminal milestone in the overall effort to:

- address the issue of soil properties relevant in routine geotechnical engineering practice in a single, comprehensive document;

- promote the greater use of in-situ testing in routine practice by discussing many of the devices, both old and new, available at the time;

- link the outcomes from in-situ testing, calibration-chamber testing, and conventional laboratory testing to produce an array of theoretical and empirical relationships for the index properties, stress state, compressibility (stiffness or modulus), and shear strength of soil; and

- illustrate the generic application of these synthesized outcomes to a wide variety of applications in practice.

Upon reviewing the contents of this report, the writer recognized not only the potential analytical power of the numerous algebraic relationships contained therein for detailed, comprehensive soil-property estimation with modest exploration effort but also the opportunity these relationships and concomitant results presented to integrate site characterization and foundation-engineering analysis and design into a seamless, iterative process. A summary of the writer's thought process behind this integration was most recently outlined and summarized in Horvath (2011).

It is worth noting that the concept of better unifying and integrating the site characterization and geotechnical analysis or design process is not unique to the writer. The subject has recently gotten the attention of others (e.g. Salgado and Fox 2010, Mayne 2012, DeGroot 2013).

However, the writer's first efforts at documenting the concept of integrating site characterization and foundation engineering go back much farther in time than 2011. What eventually became an ongoing, long-running (20-plus years) research effort at Manhattan College (eventually formally called the Coupled (later Integrated) Site Characterization and Foundation Analysis Research Project) began in the early 1990s and was directly motivated by a prediction symposium for shallow-foundation (spread-footing) settlement and bearing capacity that occurred in 1994 (Gibbens and Briaud 1994). The writer's analytical work related to this event (summarized in Horvath (1994)) primarily involved development of a site-characterization algorithm for coarse-grain soils that required as input only the most basic field data (primarily CPT q_c or SPT N_{60}) that would be available even for relatively small, low-budget projects. The outcomes from this algorithm were a comprehensive

[87] This document has since been made available to the public in digital-file format at no cost. The URL link for this is provided in the Reference section of this report.

assessment of index, stress-state, and engineering (compressibility and strength) properties under the conditions existing at the time of CPT and/or SPT performance.

Although this analytical algorithm can, in principle, be solved by manual calculation using an ordinary hand-held calculator, the large number of raw-data points that would typically be assessed in any practical application (especially if CPT as opposed to SPT data were used) make numerical solution using a digital computer a practical necessity. This can be accomplished in any number of ways. The writer chose to create and use a purpose-built code in Fortran programming language named *HINT* for this purpose.

Subsequent to 1994, both the original basic site-characterization algorithm (hereinafter referred to as Version 1.0) and, concomitantly, *HINT* have undergone aperiodic updates as new empirical relationships became known to the writer. Also, the application examples have either been improved or new ones added. The following is a chronological summary of the writer's published work along these lines subsequent to 1994:

- The subject of shallow-foundation bearing capacity was revisited and explored in much greater detail in Horvath (2000a, 2000b) with the goal of improving the agreement between calculated and measured results. As is well known, the gross ultimate bearing capacity, q_{ult}, calculated from traditional bearing-capacity theories such as those of Hansen, Meyerhof, Terzaghi, and Vesic under drained-strength conditions is very sensitive to the assumed value of the Mohr-Coulomb angle of internal friction, ϕ. These reports in 2000 explored a new methodology developed by the writer for iterating to arrive at an appropriate value of $\phi_{peak/secant}$ under the relevant operative-stress condition at bearing failure, a concept discussed generically and for broader applications by Kulhawy and Mayne (1990). The reason that an iterative approach is necessary is that the Mohr-Coulomb failure envelope for ϕ_{peak} is curved but traditional bearing-capacity solutions all require a single value for ϕ. An efficient way to deal with this is to estimate a secant value for ϕ, an approach that was suggested in concept in Kulhawy and Mayne (1990).

- The application of axial-compressive geotechnical ultimate resistance for driven piles was first addressed in Horvath (2002). The concept of using an appropriate value of $\phi_{peak/secant}$ based on operative-stress conditions was again employed for calculating both shaft and toe resistances as well as for the third capacity mechanism of cylindrical cavity expansion as a better model for tapered piles. The specific analytical theory for estimating the capacity component of the tapered portion of a pile was called the Modified Nordlund method by the writer. It is relevant to note that subsurface conditions and piles at JFKIA were used as the source data for the research presented in Horvath (2002).

- The first update to the basic site-characterization algorithm, Version 2.0, was described in Horvath (2003a). This involved a change in the empirical equation used to estimate the horizontal effective overburden stresses. This report included an illustration of how results between versions 1.0 and 2.0 compared, again using data from JFKIA.

- The next update to the basic site-characterization algorithm, Version 2.1, occurred just a year later, in 2004 (Horvath 2004). This involved a change in the assumption as to which value of the Mohr-Coulomb angle of internal friction, ϕ (peak, ϕ_{peak}, versus constant-volume/critical-state, ϕ_{cv}), was appropriate to use for evaluating the normally-consolidated coefficient of lateral earth pressure at rest, K_{onc} (= 1 - sin ϕ). Previous versions (1.0 and 2.0) of the algorithm used the former assumption whereas subsequent '.1' versions beginning with 2.1 use the latter assumption. This report included an illustration of how results between several versions compared, again using data from JFKIA.

- Although it contained no algorithm updates, Horvath (2011) is notable because it provided a concise, updated presentation of the then-current algorithm (Version 2.1) together with a worked example of spread-footing bearing capacity to illustrate the entire integrated process.

Algorithm Version 3.1

The writer's experience to date is that the primary factors affecting the outcome of the basic site-characterization algorithm, at least for coarse-grain soil, are:

- the empirical equation used to estimate the horizontal effective overburden stress, σ'_{ho}, and

- the assumption (ϕ_{peak} versus ϕ_{cv}) for calculating K_{onc}.

Using the version nomenclature of 'Version X.Y' defined in Horvath (2004), the former factor is defined by 'X' and the latter by 'Y'.

Based on the writer's experience and assessments to date as summarized in Horvath (2004), the empirical relationship used to estimate σ'_{ho} appears to have the greater effect on the calculated results compared to the assumption involving ϕ. Therefore, when the writer only recently (in 2013) became aware of a published (Mayne 2006) revised empirical relationship for σ'_{ho} it was decided to implement this new relationship even though it differed only slightly from the one the writer implemented in Horvath (2003) as Version 2.0 and used a year later as Version 2.1 in Horvath (2004). Consistent with the aforementioned version nomenclature, this latest updated algorithm is denoted Version 3.1 and is the version used for all site-characterization analyses presented in this paper. Assessment of this latest algorithm version (3.1) using the program *HINT* confirms that calculated results are virtually identical to those obtained for Version 2.1 as illustrated in Horvath (2004).

Future Algorithm Changes

As stated at the conclusion of Horvath (2002):

> "*Site characterization is clearly the key component of the proposed analytical methodology. Therefore the various correlations and algorithms used and presented herein should be updated on an ongoing basis to take advantage of the latest developments in this regard..*"

The writer has always kept this admonition in mind as evidenced by the aforementioned updates to the original (Version 1.0) algorithm from 2002 to the present Version 3.1.

In this vein, and with further regard to the selection of the appropriate value of ϕ for calculating K_{onc} as discussed in the preceding section, it is relevant to note that for some years now the writer had settled on using only ϕ_{cv}, not ϕ_{peak}, for making this calculation. This decision was based on several references cited in earlier publications by the writer that all indicated this was the more-appropriate choice.

However, it is most interesting to note that all of the recent publications by Mayne cited throughout this paper that are related to soil-property relationships to use with CPT data use the peak, not constant-volume, value for ϕ which was the original position taken years ago by Mayne (and Kulhawy) as well as the writer. More importantly and convincingly, all of the results shown in Mayne's recent publications that are based on this assumption appear to show good agreement with various case-history results.

With this in mind, during the course of developing this paper it became clear that a rethinking of the writer's current position concerning ϕ as well as additional updates of the writer's algorithm to take advantage of the latest research by both Mayne and Robertson may be warranted. The particular motivation along these lines was the results presented in Figure 14 that show a consistent difference between yield stress, σ'_{vm}, values estimated using the writer's Version 3.1 algorithm and Equation 2 proposed by Mayne (2012, 2014). Recent publications by Mayne in particular show that yield stress for both coarse- and fine-grain soils is a crucial variable for estimating most other soil properties.

To further explore this difference in yield-stress values shown in Figure 14, as discussed by Mayne (2006) a useful alternative to *OCR* when investigating overconsolidation is a newer, alternative parameter called overconsolidation difference, *OCD*, that is defined as:

$$OCD = \sigma'_{vm} - \sigma'_{vo} \qquad \textbf{(18)}$$

which means that, unlike *OCR* which is dimensionless, *OCD* has dimensions of stress.

As pointed out by Mayne (2006), the attraction of using *OCD* as an alternative to *OCR* when plotting and interpreting stress-state results is that certain cases involving stress change show up more clearly with the former compared to the latter. For example and as illustrated by Mayne using case histories, if a site is overconsolidated due to past removal of soil as the result of either natural erosion or human activity (e.g. quarrying sand as in one of Mayne's case histories) then *OCD* will plot up as a constant-value line as a function of depth whereas *OCR* will plot as an exponential-looking curve that decreases in magnitude with depth.

With this in mind, the results shown in Figure 14 were used to create plots of both the traditional *OCR* (Figure 37) and the newer *OCD* (Figure 38) as a function of depth BGS. Note that in Figure 38 *OCD* has been non-dimensionalized to atmospheric pressure solely as a plotting convenience. Note also that only the CPT results and not the pseudo-CPT SPT results from Figure 14 are shown in these new figures.

Some comments concerning basic interpretation and understanding of these plots:

- The abscissa scale in Figure 37 was intentionally truncated at *OCR* = 8 (values as high as approximately 50 were calculated in the very shallow portions of the vadose zone) in order to better illustrate the differences in results throughout most of the profile.

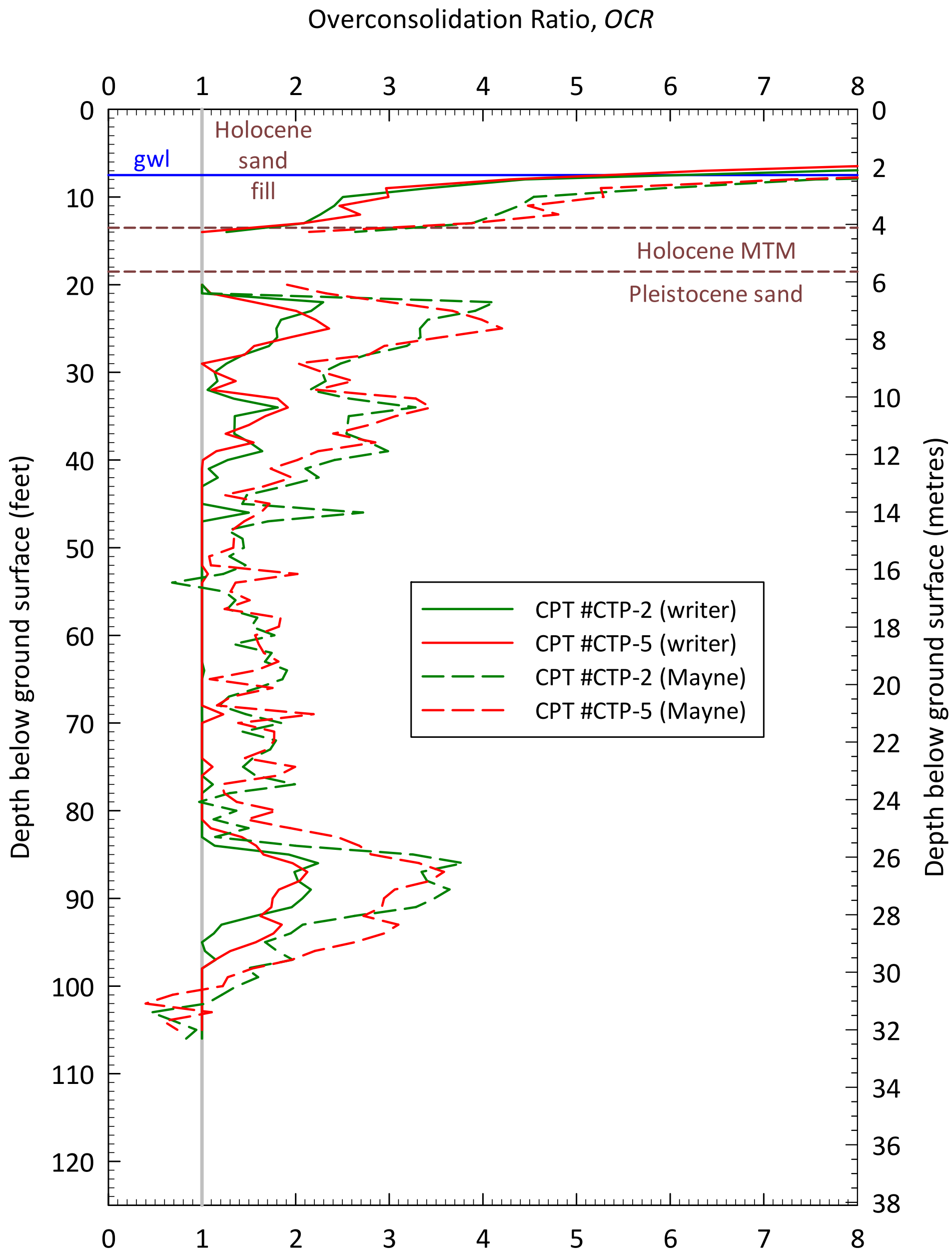

Figure 37. Overconsolidation Ratio, *OCR*, within CTA.

- *OCR* = 1 defines the normally-consolidated stress state. The writer's algorithm was constructed so that values < 1 are never calculated. This was done intentionally so as not to generate unreasonable results given the iterative nature of the writer's algorithm where results from each step and iteration in the process influence subsequent steps and iterations. On the other hand, Mayne placed no restrictions on the stand-alone empirical relationship for σ'_{vm} (Equation 2) used to calculate *OCR* so values < 1 are possible from Mayne's relationship and this is certainly the case observed in Figure 37.

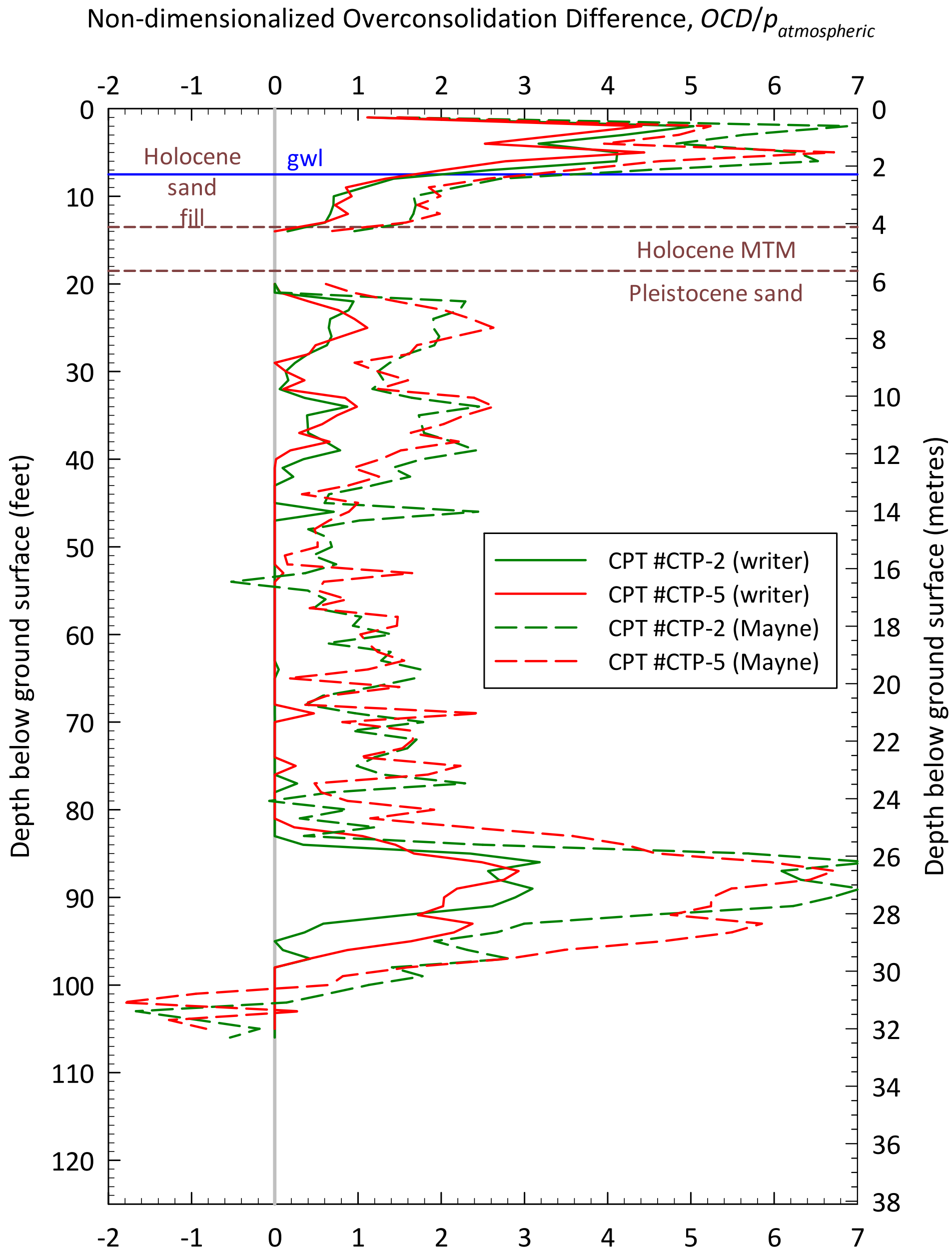

Figure 38. Non-Dimensionalized Overconsolidation Difference, OCD/p_{atm}, within CTA.

- OCD (and OCD/p_{atm} in this case) = 0 defines the normally-consolidated stress state. For the reasons explained above the writer's algorithm cannot produce values < 0 whereas values < 0 are possible with Mayne's methodology.

With regard to the specific results shown in Figure 38, it is clear that, independent of the writer's versus Mayne's results, the vertical effective stress difference reflected in the OCD is not uniform with depth within the Pleistocene sand bearing stratum. This suggests that the overconsolidation reflected in Figure 37 is due, at least in part, to a mechanism or mechanisms other than 1-D mechanical unloading, e.g. past liquefaction.

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