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ABSTRACT

Tapered driven piles have been the deep foundation of choice at the well-known John F. Kennedy International Airport (JFKIA) in New York City ever since construction of and at the airport began in the late 1940s. For many decades naturally tapered timber piles were used primarily but various brands of closed-end steel pipe piles have become preferred in recent years as design engineers have sought ever-increasing allowable axial-compressive loads per pile.

Toward the end of the 20th century, construction of new passenger terminals and a light-rail system called *AirTrain* at JFKIA pushed existing steel-piling alternatives to their performance limit in terms of both temporary driving stresses and permanent foundation loads. This led to the development of a new type of tapered steel pipe pile called the *Tapertube*™. This paper discusses the rapid evolution of the *Tapertube* pile to the degree that it is now possible to routinely install piles that have allowable axial-compressive service loads per pile in excess of 400 kips (1780 kN), with net ultimate axial-compressive geotechnical capacities per pile of the order of 1000 kips (4450 kN). This paper also discusses the results from various types of load testing performed on *Tapertube* piles at JFKIA both during and after pile driving, and compares these results to capacities calculated using a new (in 2002) analytical method that has shown great promise for use with tapered driven piles. Finally, this paper also draws conclusions and makes suggestions as to how other tools such as dynamic measurements that are routinely used with tapered driven piles might be improved to better reflect the current understanding of how tapered driven piles develop most of their axial-compressive capacity.

INTRODUCTION

Driven piles with a depth-variable circumference or perimeter over all or at least part of their length are called tapered piles. They have long been recognized as the most cost effective driven-pile alternative in applications calling for "friction piles", especially in coarse-grain soil conditions [Peck 1958].

Work at the Manhattan College School of Engineering Center for Geotechnology (CGT) was initiated in recent years to further the state of knowledge with respect to calculating the axial-compressive geotechnical capacity of tapered driven piles in coarse-grain soil [Horvath 2002, 2003]. Coincidentally, during roughly the same time frame market forces in the U.S.A. were making the first significant advance in tapered-driven-pile technology in decades. This paper attempts to link these two developments synergistically in an effort to extend and improve the states of both practice and art with regard to tapered driven piles.

BACKGROUND

Historically, the use of tapered driven piles in the New York City metropolitan area has been very common. This is due to a combination of appropriate geologic conditions (sands from Pleistocene glaciation are encountered in many areas) and local piling suppliers and contractors who have been proactive in advancing the states of practice and art for tapered piles.

In view of these factors, it is no surprise that tapered driven piles have been the deep foundation of choice ever since the well-known John F. Kennedy International Airport (JFKIA, originally named New York International ("Idlewild") Airport) in New York City was first developed in the late 1940s by The Port Authority of New York and New Jersey (PANYNJ, originally The Port of New York Authority). What has changed in the more than 50 years of construction at JFKIA is the type of tapered pile used, beginning with generic, naturally tapered timber piles and evolving through several proprietary types of closed-end steel shell and pipe piles that are filled with portland-cement concrete (PCC) after driving. A comprehensive discussion of the evolution of tapered-pile usage at JFKIA is the subject of a separate paper [Horvath and Trochalides 2004].

The very end of the 20th century saw extensive construction at JFKIA and vicinity for several new passenger terminals within the airport's Central Terminal Area (CTA) as well as for a light-rail system called *AirTrain* to connect JFKIA with nearby transit hubs. By this time frame, designer engineers were looking for piles with allowable axial-compressive service-load capacities of at least 300 kips (1335 kN). These design requirements, which pushed the edge of the deep-foundation envelope at JFKIA, ultimately and directly led to the development of a new type of tapered driven pile called the *Tapertube*™. This pile has several structural features that provide better performance under demanding driving and foundation-load conditions compared to piling alternatives that existed at that time. This is supported by the fact that allowable axial-compressive service-load capacities in excess of 400 kips (1780 kN) were eventually used for *Tapertube* piles within a relatively short time after they appeared commercially. It appears quite feasible that even larger allowable loads could be achieved in the future, especially in coarse-grain soil conditions.

PURPOSE AND SCOPE OF PAPER

The primary purpose of this paper is to trace some of the key technical steps in the evolutionary development of the *Tapertube* pile. The discussion is limited to structures at or near JFKIA which is where essentially all early evolution and usage of this pile occurred. However, it should be noted that the *Tapertube* pile has since be used on projects outside of the JFKIA area and is available for use internationally.

A secondary purpose of this paper is to illustrate the application of an interim improved analytical method for estimating the axial-compressive geotechnical capacity of driven piles, especially tapered piles, that was first reported in detail in Horvath [2002] with an important update in Horvath [2003]. An overview of this analytical method is given in this paper as all of the *Tapertube* piles discussed herein were analyzed using the updated version of this method. The calculated results are compared to measured capacities that were obtained using a variety of techniques including conventional static load tests as well as the quasi-static *Statnamic*™ and dynamic *Pile Driving Analyzer*® (*PDA*) and *CAPWAP*® techniques.

GEOLOGIC AND SUBSURFACE CONDITIONS

Despite the relatively large physical area covered by JFKIA (almost 8 mi^2 (20 km^2)), the overall geologic setting and subsurface conditions are quite uniform. A general description can be found in York et al. [1994] and is synopsized here.

Figure 1 was taken from Horvath [2002] and illustrates typical subsurface conditions within the CTA where some of the piles considered in this paper were driven. Also shown are Standard Penetration Test (SPT) field N values, N_f, and cone penetrometer (CPT) tip resistances, q_c, (the latter normalized to atmospheric pressure, p_{atm}) that are typical of conditions within the CTA. Note that the assumed SPT hammer efficiency of 45% shown in this figure is representative of SPT driving systems used up to ca. 1990 when this particular boring was drilled. The SPT driving system used in recent years in this area has an efficiency of the order of 60% as verified by field measurements.

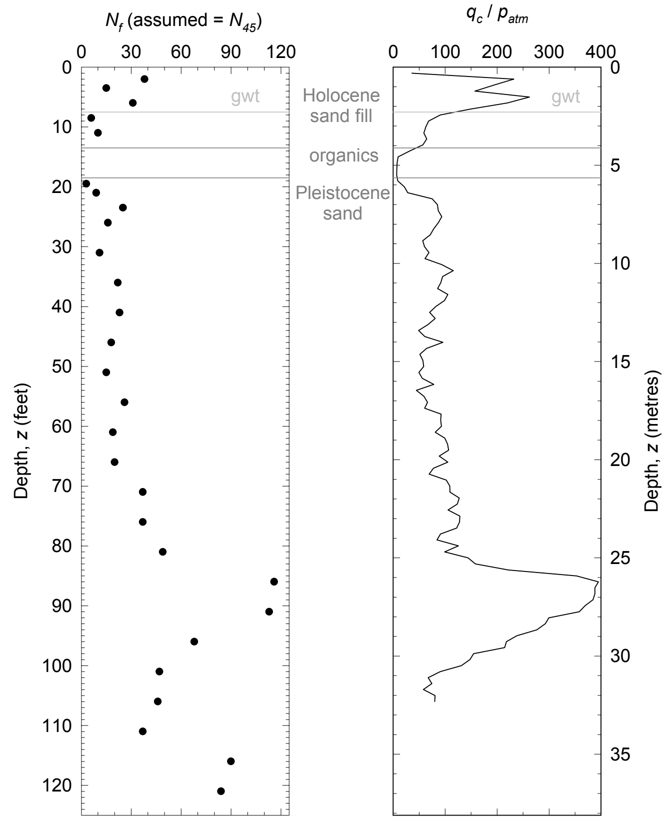

Fig. 1. Typical JFKIA CTA Subsurface Stratigraphy and In-Situ Test Results.

Virtually the entire airport property was originally a marine tidal wetland bordering on Jamaica Bay which is part of the Atlantic Ocean. Within the depth of interest for foundation purposes (approximately 100 ft (30 m)), the original Holocene wetland soils (mostly organic clay with some peat) are underlain by a stratum of sand (predominantly fine but grading coarser with depth) that is a kame (outwash) glacial-drift deposit from the recent Pleistocene glaciation that terminated several miles (kilometres) north of the airport. The current JFKIA property was developed in the 1940s by dredging similar sands from within Jamaica Bay and hydraulically pumping them over the wetland. The resulting surface topography is quite flat. Some of the piles discussed in this paper are located at or beyond the northern edge of the JFKIA property, at or just north of the former shoreline of Jamaica Bay. The Holocene organic stratum in these areas becomes very thin and eventually disappears entirely. The Holocene sand fill also disappears entirely north of the airport property. In addition, the Pleistocene sand stratum becomes both overall denser and coarser in gradation as it approaches and eventually transitions into the terminal-moraine glacial-drift deposits north of the airport.

As shown in Fig. 1, the current ground-water table is located within the Holocene sand-fill stratum. The piezometric level within the underlying Pleistocene sand stratum is close to that in the Holocene fill and was assumed so for all calculations reported in this paper.

EVOLUTION OF THE *TAPERTUBE* DESIGN

Although *Monotube* piles had been driven at JFKIA as early as 1972, it was not until ca. 1990 that they emerged as the pile of choice there [Horvath and Trochalides 2004]. The *Monotube* pile is one of the oldest, if not the oldest, type of tapered steel (cold-rolled) pipe pile [Chellis 1961]. It is a closed-end pile with a tapered lower section and constant-diameter upper section that is available in a variety of sizes and wall thickness of steel. Perhaps the most notable aspect of the *Monotube* pile is its signature visual appearance which consists of series of flutes that run longitudinally along the exterior of both the tapered and constant-diameter portions of the pile.

The steel shell of a *Monotube* pile is almost always completely filled with PCC after driving. This provides additional structural strength and stiffness of the pile section. Compared to the naturally tapered timber pile which had been a fixture at JFKIA for decades, the *Monotube* pile represented a significant increase in terms of allowable load per pile.

The most significant variables for the *Monotube* pile in terms of its axial-compressive geotechnical capacity are the taper angle, ω, and length of tapered section. Taper angle is defined as the angle, typically expressed using the imperial unit of degrees, that the planar outside surface of a pile makes with respect to its longitudinal axis. Thus a constant-diameter pile has a $\omega = 0°$. *Monotube* piles come in three standard taper angles or "types":

- Type F ($\omega = 0.33°$),
- Type J ($\omega = 0.57°$) and
- Type Y ($\omega = 0.95°$).

As noted previously, by the late 1990s design requirements for allowable axial-compressive service loads per pile at JFKIA had increased to 300 kips (1335 kN). This led to use of *Monotube* piles with the thickest steel section available and a Type Y tapered lower section that varied from 8 to 18 inches (203 to 457 mm) in diameter over a length of 25 ft (7620 mm). However, field experience, especially during driving, indicated that the desired design capacities were sometimes challenging to meet. Unfortunately, there was no available alternative due to constraints of available sizes of the *Monotube* pile. This led to the development of the *Tapertube* pile.

The *Tapertube* pile is identical to the *Monotube* in terms of its overall components, shape and geometry, i.e. a tapered lower section and constant-diameter upper section. However, there are some important structural differences between the two piles. The *Tapertube* is made entirely of hot-rolled steel components. The tapered lower section consists of a steel plate that is bent so that it has 12 flat faces or sides to create an approximately circular cross section. The constant-diameter upper section is a section of standard "pipe" pile. Overall, these features give the *Tapertube* a stronger and stiffer structure that, as field experience has demonstrated, is more robust than the *Monotube* when demanding driving and foundation-load conditions are involved.

Figure 2 shows several *Tapertube* piles assembled and ready for installation at a JFKIA job site. All are of the "Type II" design (defined subsequently) that was used for all "production" piles on the various JFKIA projects. Note that the tapered lower sections are to the right in this photo. The constant-diameter upper sections, which are spiral-weld pipe in this case, are to the left.

Fig. 2. Type II Tapertube Piles Stockpiled at Job Site and Ready for Installation.

It is of interest to note that this is not the first time that engineers in the New York City area have tried to improve upon the *Monotube* pile. In the early 1990s, at a site not far from JFKIA, engineers used a *Monotube* tapered lower section with a standard, generic pipe-pile upper section and achieved net ultimate axial-compressive geotechnical capacities in excess of 400 kips (1780 kN) [Brand 1997]. However, the *Tapertube* pile represented the first all-new tapered steel pile design in decades.

As with most new products, the *Tapertube* went through several evolutionary versions before settling on what is, at the time this paper was written (July 2003), the current production version. The two major versions of this pile are referred to in this paper as Type I and Type II. Note that at the time this paper was written there was no standard nomenclature (as there is for the well-established *Monotube* pile) for the dozens of variations (in terms of taper angle, length of tapered section, diameter, and thickness of steel) of the *Tapertube* pile that are cataloged by its distributor. Therefore, arbitrary "type" designations for the *Tapertube* pile were created by the authors and are used in this paper solely to identify variations in Tapertubes that were used on the projects described in this paper.

What distinguishes the Type I and II *Tapertube* is the detail for the connection between the top of the tapered section and bottom of the constant-diameter section. The Type I piles used what can best be described as an oversized cast-steel collar to connect the two sections. The diameter of the collar was approximately 1 inch (25 mm) greater than the diameter of the constant-diameter section above it. Subsequently, there was a concern that the oversized collar might reduce the lateral earth pressures and thus side friction along the constant-diameter upper section of the pile. As a result, a proprietary, patented connection detail was developed that eliminated the collar. A close-up photo of this improved connection detail is shown in Fig. 3. The tapered lower section is to the right and the constant-diameter upper section to the left in this photo. Piles with this revised connection detail are referred to herein as Type II and can be considered the current production version of the *Tapertube* pile.

Fig. 3. Tapertube Pile with Improved (Type II) Connection Detail.

The 12-sided "circle" of the tapered lower section that is the signature visual detail of the *Tapertube* pile is also clearly visible in Fig. 3. Note, however, that the upper end of the tapered section is bent at the factory to a true circular shape so that it fits snugly inside the constant-diameter pipe of the upper section. A weld seals the connection.

There was another evolutionary aspect of the *Tapertube* design that is unique to the work at and near JFKIA that is the focus of this paper. This has to do with the taper angle and length of tapered section used. Design requirements for the piles in question dictated that the constant-diameter upper section have an outside diameter of 18 inches (457 mm). The initial *Tapertube* pile design, designated Type Ia in this paper, used a 15-ft (4572-mm) long tapered section which resulted in a taper angle, ω, = 1.6°. This is noteworthy because this was (and still is) a significantly larger taper angle than available in the *Monotube* product line. As noted previously, the Type Y *Monotube* with ω = 0.95° was the pile being used at JFKIA at the time the *Tapertube* Type Ia pile was first introduced and used. However, use of this larger taper angle was not continued on the work at JFKIA and the next step in the evolution of the *Tapertube* design, designated Type Ib in this paper, replicated the 25-ft (7620-mm) long tapered section and ω = 0.95° of the Type Y *Monotube* it was competing against. Finally, the Type II *Tapertube* design emerged with the improved connection detail described and illustrated above. The Type II design retained the length (25 ft (7620 mm)) and taper angle (0.95°) of the Type Ib. The Type II *Tapertube* was used for the remainder of the work at JFKIA and vicinity although the allowable axial-compressive service load per pile was eventually increased to in excess of 400 kips (1780 kN). In some case, net ultimate axial-compressive geotechnical capacities per pile of the order of 1000 kips (4450 kN) have been measured.

METHODS FOR PILE-CAPACITY DETERMINATION

Introduction and Overview

Before presenting and discussing the measured and calculated capacities of the *Tapertube* piles, it is both useful and important to summarize and briefly describe the various capacity-determination methodologies that were employed. They broadly fall into two categories:

- in-situ load testing of full-scale piles and
- calculation.

Load Testing

Static Load Test. The static load tests performed for the piles considered in this paper were traditional maintained load (ML) tests, mostly using dead-weight reaction but in some cases using uplift-pile reaction. There is a tendency to view results from ML tests as "the answer", i.e. the absolute, single-valued geotechnical capacity of the pile. In reality, there are many reasons involving both the test procedures themselves as well as the interpretation of the measured load-settlement curve that make pile-load-test results really more of a range of capacities and only at some point in time at that. A detailed discussion of all the variables affecting the results and interpretation of traditional static load tests is beyond the scope of this paper but can be found in Horvath [2002].

The most significant aspect of static load testing from the perspective of this paper is the method used to measure the load applied to the top of the pile. It is now well recognized that the most-common reliable way to do this is by use of a load cell that is independent of the mechanism used to apply load to the pile [Fellenius 1990]. However, it is not uncommon, at least in U.S. practice, to rely solely on the pressure gauge associated with the hydraulic jack that is used to apply load to the pile. This is done purely for economic reasons. It is well documented that loads indicated by these gauge readings are of the order of 10 to 20% greater than the actual loads reaching the top of the pile due to piston friction within the jack [Fellenius 1990]. This is important because use of loads determined from the jack pressure gauge always results in an error on the unconservative side, i.e. a pile is always indicated as carrying more load than it really is. As will be seen, this fact is relevant for two of the six piles considered in this paper.

Quasi-Static Load Test. A relatively new and certainly novel alternative to the traditional static load test is a proprietary testing technology called *Statnamic*. A detailed discussion of this technology is beyond the scope of this paper. However, the methodology can be briefly described as essentially a dynamic one in which a mass placed over the top of a deep-foundation element is accelerated upward by igniting a propellant, imparting a downward reaction to the top of the deep-foundation element in the process. Note that a geotechnical failure (in compression in this case) is not assured using this procedure any more than it is when using a traditional static load test.

The overall *Statnamic* testing methodology, although fundamentally dynamic, can be interpreted as a quasi-static event by analytically removing the dynamic components of the applied force. Therefore there is an element of subjectivity and interpretation (which implies a potential source of error) involved in producing the desired equivalent static-load-test final result. That having been said, the *Statnamic* method has been in existence for some years now, and researched and calibrated against traditional static load tests. Therefore, the interpreted equivalent static results can be assumed to be reasonably consistent with results expected of traditional, "true" static tests such as the ML test described above. As it turns out, this was correct for the one *Statnamic* load test considered in this paper.

Dynamic Measurements. Separate from the *Statnamic* test are a variety of methodologies involving measurements made on deep foundations when subjected to shorter-duration dynamic loading conditions. This typically involves pulsing some type of wave through a pile or hardened PCC of a drilled shaft. Note that geotechnical failure in compression is never assured using any dynamic method. Therefore, the axial-compressive geotechnical resistance deduced using a dynamic method may or may not represent the net ultimate capacity of the deep foundation.

In the case of driven piles such as considered in this paper, dynamic measurements are made during driving using the stress waves created within a pile by the actual driving. These stress waves are measured and interpreted using proprietary hardware and computer software. This process involves attaching transducers to the pile near its top, and making and interpreting data in the field real-time during driving using a device such as the *PDA*. In some cases, selected data can be interpreted further in an office environment using a computerized interpretive methodology called *CAPWAP*. A thorough introductory description of both the *PDA* and *CAPWAP* methodologies can be found in Hannigan [1990]. Note that both of these methodologies were developed originally in the 1960s and have undergone extensive refinement and updating over the years to both the hardware and software components. Therefore, results obtained at some time in the past using these methodologies are not necessarily indicative of what can be produced in the present.

It is worth noting that over the years use of both the *PDA* and *CAPWAP* has been broadened from just during initial driving (with "end-of-initial-driving" (EOID) results usually of greatest interest) to include "restrike" driving at some time after EOID. The purpose of restrike driving is to evaluate the change in axial-compressive geotechnical pile capacity as a function of time. While the time dependency of deep-foundation capacity has long been appreciated in fine-grain soil, the time dependency in coarse-grain soil has only been recognized more recently and is still not fully understood, at least for driven piles [York et al. 1994, Chow et al. 1997, Horvath 2002]. It is relevant to this paper to note that significant capacity gain with time has been observed for piles, including tapered piles, driven at JFKIA [York et al. 1994]. However, all *PDA* results and *CAPWAP* analyses performed for the piles discussed in this paper were made under classical EOID conditions.

Capacity Calculation

Calculation methodologies related to the axial-compressive geotechnical capacity of driven piles fall into two broad categories:

- the "dynamic approach" using the one-dimensional "wave equation" model and solution. This essentially links the resistance encountered during initial pile driving to the long-term static capacity of the pile after driving; and
- the "static approach" based on analyses using soil-mechanics principles to model the pile after its installation in the ground and estimate its long-term static capacity.

Note that the use of "dynamic formulas" as an option under the dynamic approach has intentionally been omitted from this discussion. It has long been demonstrated that these formulas are based on a physical model and assumptions that simply never exist during pile driving. As such, their continued use cannot be defended, no matter how easy and simple (and thus attractive) it may be.

Only the static approach was used for the piles considered in this paper. The specific analytical method employed was a previously published, practice-oriented methodology developed as part of the CGT Integrated Site Characterization and Foundation Analysis Research Project [Horvath 2002, 2003]. A companion study of a variety of tapered piles driven at JFKIA over a 30-year period indicates that this method provides superior predictive accuracy compared to other, existing methodologies, at least for pile capacities soon after driving [Horvath and Trochalides 2004]. This analytical method makes extensive, formal use of soil properties developed using modern site-characterization correlations for CPT and SPT data, and applies them with an analytical procedure that attempts to better represent the cavity-expansion capacity mechanism of tapered piles. Note that the true way in which tapered piles develop their axial-compressive geotechnical capacity was convincingly established by Kodikara only in the early 1990s [Kodikara and Moore 1993]. His worked identified what is now formally recognized as a third capacity mechanism (cylindrical-cavity expansion) for deep foundations in addition to the traditional capacity mechanisms of side friction and end bearing.

PRESENTATION AND DISCUSSION OF RESULTS

Ultimate Geotechnical Capacity in Axial Compression

Table 1 contains a comparison of calculated and measured (in most cases using multiple methodologies) axial-compressive geotechnical capacities (net ultimate unless noted otherwise) for six *Tapertube* piles of all three types (Ia, Ib and II) discussed previously.

Table 1. Calculated versus Measured Pile Capacities

Tapertube Type/ Pile Reference No.	Net Ultimate Axial-Compressive Geotechnical Capacity, in kips (kN)				
	calculated	measured			
		ML test	*Statnamic*	*PDA*	*CAPWAP*
Ia/2	630 (2804)	650 (2893)	-	578 (2572)	535 (2381)
Ib/10	811 (3609)	850* (3783)	800* (3560)	624 (2777)	562 (2501)
II/3	673 (2995)	760*** (3382)	-	553 (2461)	-
II/373	521 (2318)	640**** (2848)	-	-	-
II/3.1	709 (3155)	900** (4005)	-	620 (2759)	-
II/3.2	758 (3373)	900* (4005)	-	698 (3106)	-

Table Notes
* Geotechnical failure not achieved at this load.
** Pile appeared to be on verge of geotechnical failure at this load.
*** Actual pile load likely 630 to 690 kips (2804 to 3071 kN).
**** Actual pile load likely 530 to 580 kips (2359 to 2581 kN).

Some comments are required to assist in understanding and evaluating the results shown in Table 1:

- Geotechnical failure in the ML test was interpreted using the method proposed by the first author in Horvath [2002]. It is essentially based on an assumption as to when the tip has undergone a bearing-capacity failure. For the piles considered in this paper, the required settlement measured at the top of the pile for tip bearing failure to have occurred is of the order of 1.5 inches (40 mm). As noted in Table 1, not all piles considered achieved this level of settlement.
- Available information indicates that the applied load in the ML tests for Type II pile Nos. 3 and 373 was measured only using the jack pressure gauge. Thus the indicated loads are likely to be too high for the reasons discussed previously. More likely maximum loads actually applied to the top of each pile are given in the appropriate footnotes and these suggest excellent agreement with the calculated capacity in each case.
- Type II pile Nos. 3.1 and 3.2 were each the center pile of a closely spaced five-pile group (cluster) at the time the ML test was performed. Also, in each case they were the third pile driven at the time the *PDA* measurements were made. Thus the pile capacities determined at the time of both the ML tests and *PDA* measurements likely benefited from the installation of subsequent and/or prior piles. The cumulative beneficial effect on lateral earth pressures, and, as a result, axial capacity, when driving closely spaced piles in most coarse-grain soil conditions is well known [Poulos and Davis 1980]. The current version of the calculation method used for the results shown in Table 1 is for "stand alone" piles and thus does not reflect any increase in axial capacity due to group effects.

Taking all issues into consideration, the following comments are drawn with regard to the results shown in Table 1:

- Although not shown, the interpreted equivalent static load-settlement curve based on the one *Statnamic* test performed on the Type Ib pile (No. 10) agreed very well with the actual measured results in the ML test.
- All *PDA* and *CAPWAP* capacities were significantly lower than measured capacities. Even though these dynamic measurements were made only for EOID conditions, in all cases the ML tests were performed relatively soon after driving. Therefore, although there may have been some capacity gain by the time the ML tests were performed it was likely not enough to explain the relatively large difference between dynamic and static (ML test) capacities. This raises the question as whether or not the analytical model and algorithm on which the *PDA* and *CAPWAP* capacity estimates are based properly captures the unique load-capacity mechanism of cylindrical cavity expansion exhibited by tapered piles.
- Overall, the interim improved analytical method for calculating the static capacity of tapered piles that was first presented in Horvath [2002] and updated in Horvath [2003] provides good correlation with measured capacities. The obvious exceptions are for the Type II piles (Nos. 3.1 and 3.2) that were in the center of a group. As discussed above, the present formulation of this

analytical method does not account for the increase in lateral stresses caused by driving multiple piles in a relatively closely spaced group. However, this is a capability that could be added based on future research.

Load versus Settlement

One of the ancillary capabilities of the interim improved analytical method used to develop the calculated results shown in Table 1 is the ability to generate a theoretical load-settlement curve, i.e. a simulated load test. The "curve" is actually a series of line segments connecting a series of points defined on the following basis:

- Zero load and pile settlement at the origin initially.
- The load corresponding to the peak side resistance of both the constant-diameter and tapered portions plus 10% of the peak tip capacity. This occurs at a downward movement of the top of the pile equal to the theoretical elastic compression plus 0.12 inches (3 mm).
- The load corresponding to the constant-volume (critical-state) side resistance of the constant-diameter section, the peak side resistance of the tapered section plus the peak tip capacity. This occurs at a downward movement of the top of the pile equal to the theoretical elastic compression plus 15% of the pile tip diameter (1.2 inches (30 mm) for all piles considered in this paper).
- When all load is removed, there is a net settlement equal to 15% of the pile tip diameter (1.2 inches (30 mm) for all piles considered in this paper).

Figure 4 shows the typical results obtained using this procedure for one pile (Type Ia pile No. 2 in Table 1).

CLOSING COMMENTS

The *Tapertube* pile represents the first significant advancement in tapered-driven-pile design in decades. Because it is fabricated from hot-rolled steel, this pile does not have the manufacturing limitations of cold-rolled products. This opens the possibility of using the *Tapertube* pile for allowable loads per pile well in excess of what was previously thought to be achievable with tapered steel pipe piles. This potential is particularly intriguing for marine applications which have, historically, seen relatively little use of tapered piles.

It appears that the interim improved analytical method for tapered piles first presented by Horvath [2002] and recently updated in Horvath [2003] offers promise not only as a design tool in project-specific applications but for broader research purposes to optimize tapered-pile design. It is clear that further study centered around taper angle is desirable to optimize pile design not only at JFKIA but in any application of tapered piles. This is because the majority of the axial-compressive geotechnical capacity of a tapered pile comes from its tapered portion. Thus future research into optimizing tapered-pile design should consider the variables of:

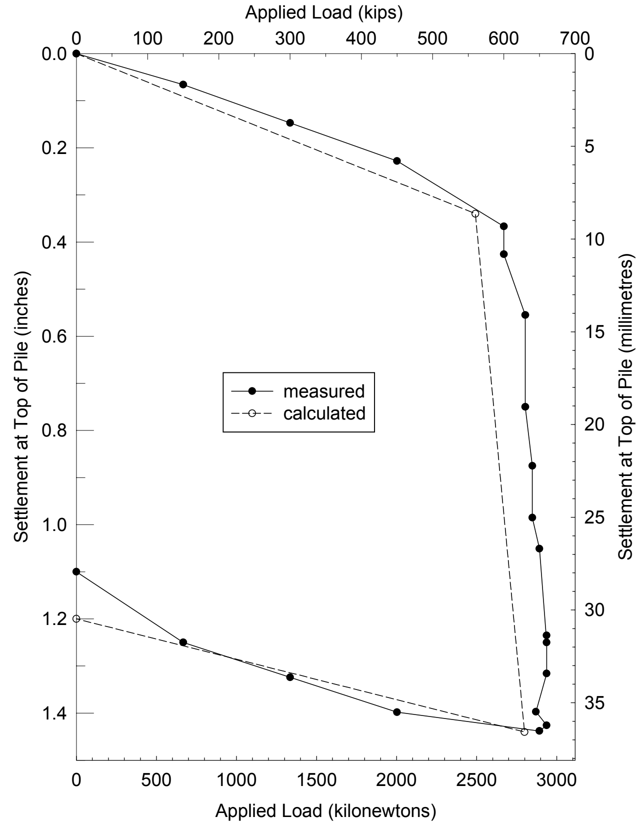

Fig. 4. Measured versus Calculated Load-Settlement Curves.

- taper angle,
- length of tapered section and
- depth of embedment of the tapered section (due to its effect on vertical effective overburden stresses).

Additional discussion about how this might be achieved is presented in Horvath and Trochalides [2004]. As noted above, it would also be useful to extend this analytical method to consider the effects of driving multiple piles in a group or cluster as well as to take into account time-dependent effects.

Finally, it is important to note that analytical methods alone are not sufficient for successful driven-pile installation. Field observation and measurements play an important role in complementing office analyses. To that end, dynamic measurements using the *PDA* and *CAPWAP*, whether for EOID or restrike conditions, are a well-established tool in routine practice. Therefore, it is suggested that the analytical algorithms built into these dynamic-measurement tools be reassessed to see if they properly model what is now understood about how tapered piles develop most of their axial-compressive capacity, from the mechanism of cylindrical cavity expansion. It is important to remember that the *PDA*/*CAPWAP* methodologies, and the wave equation solution on which they are based, were all developed beginning in the 1960s. Therefore they only consider the traditional deep-foundation capacity mechanisms of side friction and end bearing. It was not until the 1990s that the third capacity mechanism of cylindrical cavity expansion that governs the tapered portion of a deep foundation was clearly defined and established. Consequently, it would appear appropriate to revisit the wave equation and all the methodologies and technologies such as the *PDA* and *CAPWAP* that derive from it to better model the behavior of tapered piles during driving.

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DISCLAIMER

Reference in this paper to tradenamed products, procedures and equipment is solely for informational purposes and does not constitute an endorsement of said products, etc. by the first author, the CGT or Manhattan College. No financial compensation was made to the first author, the CGT or Manhattan College by UFC or any other outside party involved in the work reported herein.

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