Advancements in Rapid Load Test Data Regression

by

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Advancements in Rapid Load Test Data Regression

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ABSTRACT

Rate-dependent effects introduced during rapid and/or dynamic events have typically been oversimplified to compensate for deficiencies in present analyses. As load test results are generally considered as the basis of performance from which foundations can be designed, it is imperative that the analyzed load test data be as accurate as possible. In an attempt to progress the state of load test data regression, this dissertation addresses two common assumptions made during the regression process: (1) the statnamic damping coefficient is constant throughout the entire load test and (2) the concrete stress-strain relationship is linear-elastic. Also presented is a case study where the inherent features of a rapid load test proved useful in identifying the occurrence and proximity of a structural failure within a drilled shaft.
1.0 Introduction

In the past 20 years, rapid load testing has developed into a formidable option for evaluating foundation performance. Prior to rapid testing, large foundations were either slowly loaded using a hydraulic jack placed between the foundation and a reaction system or dynamically loaded via a hammer. Both systems had their advantages and disadvantages. In fact, the two test methods diametrically opposed each other in almost every aspect from economy to reliability. Eventually, the ever-increasing load demands from newer structures surpassed the ability of these traditional test methods to fully mobilize a foundation. The need for higher load-range testing combined with the insufficiencies of existing test methods ushered the onset of rapid load testing.

Since its inception, rapid load testing has proven to be more cost-effective than traditional static testing and more reliable than dynamic testing. However, uncertainty still surrounds some of the parameters necessary for determining the foundation resistance. Similar to other fields of engineering, reasonable assumptions are made to simplify the analysis and arrive at an answer that may not be entirely correct but is typically regarded as acceptable. To date, these assumptions have proved sufficient under most circumstances. But in a continual effort to progress load testing technology, these assumptions must be scrutinized to determine whether they significantly affect the final foundation capacity. This dissertation investigates two such assumptions and highlights an additional benefit inherent to rapid load tests.
Chapter 2 focuses on a particular data regression method, the Unloading Point Method (UPM) (Middendorp, 1992), and an assumption regarding the system damping made therein. Chapter 3 addresses the assumed linear-elastic behavior of concrete and the disregard for rate effects when analyzing a pile segmentally. Chapter 4 presents a case study where a rapid load test proved useful in detecting structural failures that would otherwise go undetected.
2.0 Laboratory Statnamic Testing: A Numerical Modeling Approach

The damping associated with dynamic or statnamic tests has typically been over simplified to provide a "catch-all" factor for those responses that could not be fully accounted for using present analyses. This factor has historically been considered constant for a given pile - soil system, but recent investigations as well as previous case studies show the plausibility of another explanation. This study hopes to entertain the hypothesis that damping is more closely associated with the increase in strain and not the total strain. Therein, the change in the volumetric strain is being scrutinized to investigate its relationship to damping. To validate this assumption, a numerical model is created which simulates the testing of a full-scale drilled shaft and results analyzed to determine the extent of the zone of influence and volumetric contribution to damping.

2.1 Introduction

In the area of quality assurance of foundations, numerous factors play into providing safe, reliable structures. These include, but are not limited to, pre-construction soil investigations, structural material properties, construction methods, inspection methods, and post-construction verification load testing. In actuality the latter provides proof that all the rest were perhaps adequately addressed. Rapid load testing (such as statnamic) is one such method that assures a foundation can adequately support the anticipated design loads.
Rapid load tests (ASTM 7120) are categorized as being faster than static testing (ASTM 1143) while being longer in duration than dynamic testing (ASTM 4945). The significance lies in the analysis of the foundation response to such a loading. The rapid nature of the test induces foundation acceleration, but the relatively long duration when compared to dynamic tests simplifies the regression as no wave mechanics analyses are necessary. To date, single degree of freedom systems have adequately modeled the foundation response so as to ascertain a predicted ultimate static equivalent. Refinements to these methods continue to evolve that further the understanding of all rapid loading events as well as similarly modeled dynamic events. One such refinement is presented herein.

2.2 Background

The impulsive nature of a statnamic test introduces rate-dependent components to the static response of a foundation. The equation of motion describing a statnamic event is:

\[
F_{\text{statnamic}} = kx + ma + cv
\]  

(2-1)

where \( F_{\text{statnamic}} \) is the applied force, \( kx \) is the desired equivalent static response (theoretical spring force), \( m \) is the mass of foundation and soil contributing to inertial effects, \( a \) and \( v \) are the acceleration and velocity of the foundation, and \( c \) is the damping coefficient. In almost all geotechnical applications, the damping force \((cv)\) attributed by soil is regarded as viscous in nature. All values required for solving the equivalent static response are either recorded or can be easily found save two, the spring constant \((k)\) and the damping
coefficient \( (c) \). As the foundation response is non-linear, the \( kx \) term (which is the equivalent static response) remains coupled as a single unknown. This leaves two unknowns and one equation.

One of the most common methods to solve the above equation from statnamic test data is the Unloading Point Method (UPM), or some variations such as the Modified Unloading Point, or Segmental Unloading Point (Mullins et al., 2002). These methods make two critical assumptions regarding \( kx \) and \( c \): the static capacity of the pile or pile segment is constant while plunging and the damping coefficient is constant throughout the test. With these two assumptions in place, it is possible to determine the damping coefficient, and thus the equivalent static response.

Two points on the statnamic load-displacement curve are of particular interest when performing these procedures (Figure 2-1). Point (1) at which the maximum statnamic force is achieved. This point corresponds to a point of yield (or post yield) on a theoretical static curve (1'). The second (2) is the point of maximum displacement, where the velocity of the foundation and resulting damping force \( (cv) \) equals zero. This point also corresponds to the point of maximum displacement on the theoretical static curve (2'). At that instant, the static capacity can be determined using the Eqn. 2.

\[
F_{\text{static}} = kx = F_{\text{static}} - ma
\]  

(2-2)

The value of \( kx \) is determined and assumed constant from point (1) to (2). This enables the damping coefficient to be calculated within this range. Typically, either the average or median value of \( c \) is taken, but reviewing the overall trend may lead to a more appropriate value (Transportation Research Board, 2003) (Figure 2-2).
As the UPM determines the damping coefficient between points (1) and (2), it effectively determines the ultimate capacity. However, it has been shown that at smaller displacements, in the more elastically responding regions of the loading, that same damping coefficient is not appropriate. Therein, the damping coefficient may not actually be constant throughout the entire loading event when plastic deformation is achieved. This is likely to be a by-product of the size of the zone of influence engaged by the pile at a given degree of loading; this concept forms the basis of this chapter.

2.3 Case Studies

Previous studies have shown that the statnamic damping coefficient as defined by Eqn. 2-1 is not constant throughout the entire load test, but decreases from a higher, pre-yield value. These conclusions were based on the rationale that the damping force, \( cv \), accounts for all differences between the inertia corrected statnamic force and the observed static capacity as shown in the rearranged configuration of Eqn. 2-1.

\[
    cv = (F_{\text{statnamic}} - ma) - F_{\text{static}} \tag{2-3}
\]

2.3.1 Case 1: Shallow Foundations

The first of these studies (1997) involved a series of static (SLT) and statnamic (STN) tests conducted at the Turner-Fairbanks Highway Research Center (TFHRC) in McClean, Virginia (Mullins, et al. 2000). Therein, three side-by-side footings founded in sand were tested. The first of the three was loaded statnamically with four incrementally increasing load cycles up to and beyond the bearing capacity (two cycles under-yield and
two beyond-yield). The second footing was tested statically to a displacement that fully defined failure. The third was loaded with a single statnamic load cycle that again defined the bearing capacity.

When comparing the true static (Test 2) and the derived static (Test 3) capacities, good agreement was found for the ultimate capacity but significant variation was noted in the response prior to yield (Figure 2-3). The difference between the inertia corrected load of Test 3 and the static capacity of Test 2 was then determined at each displacement and used to back-calculate the damping coefficient that satisfied Eqns. 2-1 and 2-3 for the corresponding velocity. Figure 4 shows both the inertia corrected statnamic load response and the true static as well as the back-calculated damping coefficient. The reported results indicated that the damping coefficient decreased in value by more than 50% after shear failure occurred.

Similarly, the UPM determined damping coefficients for the four cycles of Test 1 showed similar values for like displacements or degrees of loading when compared to the back-calculated. The failure envelope defined by the four cycles was slightly higher than the static envelope which was attributed to slight densification caused by the four rapid load tests.

2.3.2 Case 2: Pile Group Tests

A second case study showing discrepancies in the assumed constant damping coefficient was also conducted at the TFHRC but by Ealy and Justason in 1998 on a small scale driven pile group. The group consisted of nine, 75 mm diameter, 3.6 m long
piles arranged in a 3 x 3 configuration. Test procedures included: static quick test, constant rate of penetration, and statnamic. The purpose of the study was to show the similarities and/or differences in the load-settlement response when subjected to a range of loading rates (Ealy and Justason, 2000).

With regards to the effects of the damping coefficient selection, coefficients ranging from zero to twice the UPM-derived value were inputted to illustrate the variations in the derived static capacity. In general, very little difference could be noted in the elastic regions of the curves with more pronounced variations at post-yield displacements (Figure 2-5). The authors concluded that even though the UPM-derived static response was reasonable when compared to the static; however, the overall shape of the curves did not follow the static or the CRP test results exactly. They continued by stating that perhaps a more sophisticated damping coefficient may be appropriate to more closely match the true static response. Figure 2-6 shows similar results to that observed in Case 1 whereby the authors performed the same type of back-calculation to develop a trend of actual damping versus displacement.

2.3.3 Case 3: Small Scale Drilled Shafts

This study involved the static and statnamic testing of 3 m long, 11.4 cm diameter shafts cast in the TFHRC test pits in 2004. Therein, 8 shafts were constructed using full length temporary casing and duplex drilling (drilling and casing advancement together). The use of full length temporary casing ruled out variations in side shear from slurry properties and/or their effects. The water table was maintained below the tip of the shafts
during installation and the concrete was placed by free fall into the cased hole. A single
#4 (12 mm) reinforcing bar was centrally located which was instrumented with strain
gages for end bearing delineation. All shafts were tested statically and statnamically, but
in various orders depending on location. Most shafts were tested with three cycles, either
STN/SLT/STN or SLT/STN/SLT. This allowed subtle changes in the strength envelope
to be identified when trying to compare two tests that should have been identical.
However, in this loose sandy soil, all tests after the initial load cycle appear to fail at the
same load and displacement with a similar response. Figure 2-7 shows the comparison of
the static and statnamic testing along with the UPM-derived and back-calculated values
for the damping coefficient.

In all the cases cited, two apparent damping responses appear to have existed: a
pre-yield or pre-shearing value and a lesser post-yield value linked by a rapid transitional
period. It is at this transitional point that the radius of the zone of influence is thought to
collapse such that only the soil remaining within close proximity of the shear interface
contributes to damping. Numerical modeling which is presented later was undertaken to
shed some insight.

Since damping is caused by energy loss due to soil particle interaction as well as
the rate at which they interact, the size or volume of the soil affected by the loading event
should be directly related to damping. The energy associated with damping is lost and
not stored and therefore is dependent on soil undergoing active strain and not the
cumulative amount of strain experienced. It is hypothesized, herein, that the damping
coefficient at a given displacement is closely related to the instantaneous volume change
of the soil surrounding the foundation. Up to the point of shear failure around a pile, the radius of the affected soil and the straining volume increases; upon shearing, the more distal soils no longer continue to strain, and the radius of the zone and the volume of actively straining soil decreases. The changing volume of strained soil signifies a change in the energy absorbing potential of the soil matrix (the damping coefficient, $c$).

2.4 Impulse Response

A side issue relevant to the idea of a higher pre-yield value of the statnamic damping coefficient, but non-related to the changing volume of strained soil, is the investigation into the impulse response of a statnamic test. It is introduced herein only as additional support to the hypothesis and does not intend to offer an alternative analysis procedure but to merely serve as an indicator of damping variability. The previous discussion defines damping as an energy-related phenomenon; however, an indication of the variability of damping becomes evident when considering the statnamic event in the time domain.

The impulse imparted by the statnamic event can be characterized by the following equation:

$$\int (F_{statnamic} - ma)dt - \int (F_{static})dt = \int (cv)dt$$  \hspace{1cm} (2-4)

For the purposes of evaluation, the inertial component is grouped with the measured statnamic force, making the first term of Eqn. 2-4 (impulse of the inertia corrected statnamic force) possible to calculate, at least in its discrete or measured form. The term
on the right side of the equation represents the impulse, or momentum, lost in part to the damping force. The second term which is somewhat non-intuitive represents the impulse, or momentum, stored by the equivalent static force. Taking the derivative of Eqn. 4 with respect to top of shaft displacement lends insight into how the impulse of each force changes throughout the loading event.

\[
\frac{d}{dx} \int (F_{statnamic} - ma) \, dt - \frac{d}{dx} \int (F_{static}) \, dt = \frac{d}{dx} \int (c v) \, dt \quad (2-5)
\]

In this form, Eqn. 2-5 is particularly difficult to evaluate any further. Since it is hypothesized that the term \( c \) is not constant and likely a function of time or displacement, the damping force \( cv \) cannot be legitimately uncoupled. However, the discrete form of the equation at a given time step proves to be more manageable:

\[
c = \frac{(F_{statnamic} - ma)}{v} - \frac{F_{static}}{v} \quad (2-6)
\]

where \( c, v, a, F_{statnamic}, \) and \( F_{static} \) are all functions of time. In this form, it is apparent that the damping coefficient is composed of two distinct terms of which the first can be calculated, but the second remains unknown. Though this process may seem to have been unfruitful, plotting \( (F_{statnamic} - ma) / v \) (hereafter referred to as the impulse response) reveals the usefulness of this method (Figure 2-8).

Figure 8 represents the impulse response of the third case study previously discussed as compared to the true and UPM calculated damping coefficients. It offers an alternative to viewing the statnamic event as a load-displacement curve and represents
the graphical interpretation of Eqn. 2-6 where the difference between the impulse response and damping curves is attributed to the equivalent static component, $F_{static}/\nu$. Though the static component remains unknown, the predominant advantage of viewing the event in this manner is the ability to clearly indicate the transition between pre-yield and post-yield conditions without knowledge of the actual static capacity. Noting the similarities between the impulse response and the true damping reinforces the hypothesis of a higher pre-yield damping coefficient.

2.5 Numerical Model

In order to test the hypothesized damping variation, a numerically modeled shaft was generated and subjected to different simulated loading scenarios. The purpose of the modeling was to isolate the zone of influence around a loaded pile by determining both the radial extent and the degree of volumetric change experienced in these zones. Therein closer soils would experience more strain and more distal soils less. The total cumulation of the volume change within each soil element provides an indication of the strain energy potential. Modeling of the zone of influence and the soil parameters that affect this zone (e.g. soil type, soil strength, and relative stiffness between the pile and the soil) were initially conducted under static conditions in order to shed light on this phenomenon without compounding the complexity from inertial or damping forces. Parameters other than those directly related to the surrounding soil matrix, such as pile diameter and length, were held constant to eliminate the vast complexity of the system and focus on a general understanding of the system response. Once a clear understanding
of the soil/shaft interaction was obtained, complexity was progressively added to the model.

The computer program chosen to perform the numerical modeling was FLAC (Fast Lagrangian Analysis Continuum). FLAC is a two-dimensional explicit finite difference program that proves particularly useful in simulating the plastic flow of materials (e.g. soil, rock, and concrete) upon yielding. The materials are represented as elements that form an interlocking mesh. Once material properties are assigned to each element, various loading conditions can be applied to the matrix and the response of each element recorded (Itasca Consulting Group, Inc., 2002).

An axi-symmetric mesh containing nearly 1800 elements defined the 1 m diameter (3.28 ft) x 10 m (32.8 ft) long shaft and the zone of influence to a radial distance of 12 D (12 x shaft diameter) and depth below the toe of ½ L (½ x shaft length). Such an extreme radial boundary was chosen as to ensure total inclusion of the entire zone of influence. The matrix was discretized into small regions within close proximity to the soil/shaft interface that increased geometrically in size with increasing radial distance (Figure 2-9). The soil consisted of a nonlinear elastoplastic (hyperbolic Duncan-Chang constitutive model), homogeneous sand matrix with properties corresponding to SPT correlations found in the commentary of Shaft 1-2-3 (Mullins and Winters, 2004). Upon generation of the soil/shaft matrix, an interface was defined between the side of the shaft and the adjacent soil to allow slippage along the boundary. This interface was given properties synonymous with those assigned to the soil, since the strength of the shear interface of a drilled shaft is widely accepted as being the strength of the surrounding soil.
(soil/soil interface instead of a concrete/soil interface). Successive models were run that entertained Standard Penetration Test N-values of 20, 30, and 40 (Table 2-1).

### 2.6 Statically Loaded Model and Results

Once the insitu soil conditions for the model were established, a static load test was performed to a displacement is excess of 0.05 m (5% of the shaft diameter) to ensure full mobilization. Load was applied to the top of the shaft in increasing 20 kN increments. Figure 2-10 shows the load-displacement curves for the three different Standard Penetration N-values. In order to ensure that the load-displacement responses are reasonable, an envelope showing Shaft 1-2-3 output is plotted with and sufficiently bounds the computer-generated data.

Results from the static model show trends that are congruous to the hypothesized soil behavior (Figure 2-11). It is apparent by observing the ground surface displacement profiles that the volume of strained soil increases radially from the shaft to a point of maximum volume prior to shear failure. Once shear failure is reached, only the soil immediately surrounding the shaft significantly displaces, thereby decreasing the actively

<table>
<thead>
<tr>
<th>Table 2-1 Model Soil Properties Corresponding to SPT N-values.</th>
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<tbody>
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<td>Soil Properties</td>
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<tr>
<td>---</td>
</tr>
<tr>
<td>$\rho^a$ (kg/m$^3$)</td>
</tr>
<tr>
<td>$E_i^a$ (kPa)</td>
</tr>
<tr>
<td>$G_i$ (kPa)</td>
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<td>$K_i$ (kPa)</td>
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<tr>
<td>$\phi$ (degrees)</td>
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<td>$\nu$</td>
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$^a$ from Shaft 1-2-3 (Mullins and Winters, 2004)
affected zone, or the zone of influence. For the purposes of this article, the maximum significant radial extent of the zone of influence is determined by an arbitrarily chosen displacement value of 0.0001 m.

Interestingly, the change in the cumulative volume (total volume lost due to the straining soil mass) throughout the static loading (Figure 2-12) mimics the trend in the load-displacement curve. Furthermore, by taking the displacement-dependent derivative of the cumulative volume, a trend appears which closely resembles the damping noted by the aforementioned case studies (Figure 2-13). In agreement with the back-calculated damping coefficient in Figure 2-4 is the decreasing trend in $dV/dx$ or $V/x$, hereafter referred to as the displacement-dependent volume change (DDVC). Near the beginning of the load test, there is a large change in volume. Shortly into the loading cycle, the DDVC rapidly decays through a transition until ultimately diminishing to a near constant value well into shear failure.

Though the trends noted in the numerical model are useful in that the geometric shape of the curve provides insight into the occurrence of significantly sizeable events (point of failure denoted by sudden stability of the DDVC) throughout the loading, it is impractical, if not impossible, to capture soil displacement information at every location within the zone of influence (especially at depth). If the numerical solution is to be useful, the model data must be recorded in the same fashion as is plausible in the field. For this reason, it is necessary to investigate alternative methods of arriving at the displacement-dependent volumetric change.

An alternative method for calculating the DDVC involves recording the ground surface profile (Figure 2-14) of the surrounding soil and determining the incremental loss
of soil volume associated with the depressed surface. With the aid of current data acquisition devices and displacement transducers, this method becomes an attractive option for verifying this soil behavior during shaft loading. Given an initial soil volume where the boundaries are located at a sufficient distance away from the shaft as not to experience any effects from the loading event (Figure 2-15), it is reasonable to conclude that the incremental loss of volume at the ground surface is the change in volume experienced by the system. Indeed, a comparison of the DDVC calculated using both methods shows the similarities (Figure 2-16) and reinforces the validity and practicality of using soil surface measurements to indicate total change in soil volume. This approach was then used for all subsequent models.

2.7 Dynamically Loaded Model and Results

Once confidence was obtained in the results of the statically loaded model, a more complex loading condition was applied to the insitu conditions already established. Doing so provided a means of comparing static and statnamic load test results on two shafts of identical insitu conditions, a luxury not available during full-scale piles load tests. Dynamic effects were introduced into the computations by applying a simulated statnamic load pulse, initially in the form of a sinusoidal function, then later as an actual statnamic load pulse recorded during a typical load test.

Data from the numerical run was recorded and analyzed using the UPM (Figure 2-17). Similar to the previously mentioned research results (Figure 2-3), the UPM satisfactorily predicted the static capacity near the unloading point, but significant variation existed throughout most of the test.
The large zone bounded by the maximum statnamic force and maximum displacement afforded the opportunity to calculate the UPM damping coefficient over a large percentage of the test. This occurrence, coupled with the back-calculated damping coefficient, allowed for a representative plot of the pre-yield, transitional, and post-yield statnamic damping coefficient (Figure 2-18). The overlapping region between the true damping and UPM damping curves shows agreement between the calculations and ultimately offers a reasonable plot of a variable damping coefficient. Equipped with this plot, it was desirous to determine the cumulative volume change and its relationship to damping.

Since it was predetermined from the static test results that an idealized zone of influence could be described from the ground surface profile alone, only nodal locations along the ground surface of the model were monitored and analyzed to compute the DDVC. As before, the cumulative loss in volume resulting from the depressed ground surface profile was calculated. When taking the change in the total volume with respect to top of shaft displacement, the trend displayed notable geometric similarities to the plot of the damping coefficient (Figure 2-19).

As with the results from the static model, the DDVC from the dynamic model is large prior to yielding but decreases with the damping coefficient. Near the end of the test, both the DDVC and the damping coefficient increase rapidly to a very large value. Similar calculations were conducted for the other specified soil conditions (SPT N-values corresponding to 30 and 40).
The same trends were noted in all three insitu conditions (Figure 2-20). When plotted with the DDVC from the accompanying static tests, it becomes apparent that the loading rate has an effect on the magnitude of the DDVC. Although the magnitude of the volume change between the static and dynamic models are quite different, it is reasonable when considering that inertial resistance introduced to the soil during dynamic loading decreases the magnitude of the ground displacement. Also evident is the increase in the peak value of the statnamic DDVC with increasing SPT N-values. After shear failure, denoted by the drastic change in the corresponding static DDVC curve, the statnamic DDVC for all three tests seemingly converges asymptotically to a like value, similar to the true damping as seen in Figure 2-21. Considering the relationship noted between the DDVC and the true and UPM damping coefficients, it is presumable that the pre-yield value of the damping coefficient increases in a similar manner with increasing soil strength (SPT N-value).
Figure 2-1 UPM Time Window for $C$ Determination (Adapted from Transportation Research Board, 2003).

Figure 2-2 Variation in $C$ Between Times (1) and (2) (Adapted from Transportation Research Board, 2003).
Figure 2-3 Static/Static Derived Comparison (Adapted from Mullins et al., 2000).

Figure 2-4 Back-Calculated Damping Coefficient from Plate Load Tests (Adapted from Mullins et al., 2000).
Figure 2-5 Sensitivity of Derived Static Curve (from Statnamic) to Values of Damping Constant $C$ (Adapted from Ealy and Justason, 2000).

Figure 2-6 Theorized Variation in Damping vs. Settlement (Adapted from Ealy and Justason, 2000).
Figure 2-7 Comparison of True Damping and UPM Damping.

Figure 2-8 Comparison of True Damping and UPM Damping to the Impulse Response of the Inertia-Corrected Statnamic Pulse.
Figure 2-9 FLAC Output of Axi-Symmetric Model.

Figure 2-10 Computer-Generated Static Load-Displacement Curves.
Figure 2-11 Ground Displacement Profiles During a Static Load Test (N=20).

Figure 2-12 Cumulative Volume Change (N=20).
Figure 2-13 Displacement-Dependent Volume Change (N=20).

Figure 2-14 Modeled Ground Surface Profile from a Static Load Test.
Figure 2-15 Method of Calculating the Change in Volume Using the Ground Surface Profile.

Figure 2-16 Comparison of DDVC Calculation Methods for a Static Load Test (N=20).
Figure 2-17 UPM Derived Static Response for a Static Load Test (N=20).

Figure 2-18 Comparison of True Damping and UPM Damping Using Numerically Modeled Results (N=20).
Figure 2-19 Relationship Between the DDVC and the True and UPM Damping Coefficients (N=20).

Figure 2-20 DDVC Values for Various In-situ Conditions Calculated from Modeled Static and Statical Tests.
Figure 2-21 DDVC Values for Various In situ Conditions Compared to Their Corresponding True Damping Curves.
3.0 Concrete Stress Determination in Rapid Load Tests

The effects of loading rate on concrete strength and elastic modulus have long been recognized with regards to seismic and/or dynamic loading events. In such cases an increase in strength is reported to show up to 15 percent higher values. Rapid load tests of deep concrete foundation elements use the concrete modulus determined from field calibration via strain gages directly under the applied (known) load. Therein, rate-dependent effects can be directly incorporated into the determination of load from strain measurements elsewhere in the foundation element. However, in many cases the strain rate at various strain gage levels throughout the foundation may differ and the magnitude of strain may or may not lend itself to linearly approximated stress-strain relationships. Hence, the modulus selected to determine the load sensed at these levels can produce erroneous results if a single value is assumed. This chapter presents the findings of an experimental program designed to quantify the relationship between concrete stress, strain, and strain rate. Developed equations are used to show the effect of various stress evaluation methods on rapid load test results.

3.1 Introduction

Deep concrete foundations provide the support and stability for a vast majority of large structures built today. These foundations predominantly take the form of driven piles, drilled shafts, auger-cast piles, or some variation thereof. As designs are placing
more emphasis on the reliability of foundation elements, it is common practice to perform a load test on representative foundations. Not only can the load test results verify the structural integrity of the foundation, but also determinations can be made whether to lengthen or shorten the system in order to provide for additional capacity or reduce costs.

Regardless of the type of load test used or analysis procedure performed, the load test results are generally accepted as an established basis for the performance of the finished product. With this in mind, it is imperative that analysis of the load test results be accurate.

3.2 Background

In long test piles that exhibit a significant amount of side shear, multiple strain gages are often embedded at different levels so as to better understand the distribution of forces along the length of the pile (Figure 3-1). Strains recorded at these gage locations lend insight into the amount of load transferred to the surrounding soil. In order to perform these computations, the cross-sectional area and elastic modulus at each gage location are typically used to estimate the load at that level. In the case of the cross-sectional area, it can be accurately measured prior to driving (driven piles) or legitimately assumed based on the borehole diameter (drilled shaft). The modulus at each gage location is taken as a composite modulus based on the modular ratio and the proportions of steel and concrete.
Where the elastic modulus of steel is well documented and remains relatively constant regardless of its grade (200 GPa or 29000 ksi), information gathered from a concrete compression test (ASTM C39) on a sample of the mix and ACI equations are often used to estimate the elastic modulus of concrete. These equations are based on a secant modulus of approximately $0.45 f'_{c}$ (Eqns. 3-1 and 3-2).

$$E_c = 33w_c^{1.5} \sqrt{f'_{c}} \quad (w_c \text{ in pcf and } f'_{c} \text{ in psi}) \quad \text{or} \quad (3-1)$$

$$E_c = 57000 \sqrt{f'_{c}} \quad \text{(for normal weight concrete)} \quad (3-2)$$

Alternately, the composite modulus can be calculated directly by calibrating the measured load at the top of the pile with near-surface strain or modulus gages. This process of “matching” the top level strain to the load eliminates the need to determine the individual contribution of the steel and concrete, for the calibration inherently accounts for the composite cross-section. Adjustments are then made depending on the reinforcement ratio throughout the length of the pile. If the measured load and strain are used to back-calculate a composite modulus as a function of time, it becomes apparent that the modulus changes significantly throughout the test (Figure 3-2).

Whether using the ACI equations or near-surface gages, the composite modulus is assumed to remain constant throughout the duration of the load test, and a simple linear relationship (Hooke’s Law) is used to relate the measured strain to the stress at a particular gage level (Figure 3-3). This relationship may be valid as it applies to the steel, but the nonlinear behavior of concrete can lead to grossly inaccurate estimations of stress within the concrete portion of the composite section. A more representative estimation of stress
can be computed with the implementation of a nonlinear model where the concrete modulus varies as a function of the level of strain.

Recently, consideration has been given to the use of a variable strain-dependent modulus in the regression of static load test data (Fellenius 2001). In this particular case study, static load test data on a 20 m monotube pile was evaluated using both a strain-dependent modulus and a constant, average modulus. Results from the evaluation indicated that the mid-level pile stresses were lower than what was computed using a constant modulus, and lower level stresses were higher. It was concluded that if the constant modulus were used, the “resistance acting between the two levels would have been determined with an about 10 percent to 20 percent error.”

The Hognestad model is one of the more popular parabolic stress-strain relationships whose first derivative, or modulus, varies as a function of strain. It expresses a stress-strain curve that is only dependent on the compressive strength and the corresponding ultimate strain (Eqns. 3-3 and 3-4). Though the use of a nonlinear stress-strain model may account for a strain-dependent modulus, it does not address unloading stresses or compensate for load rate effects that occur during rapid and/or dynamic load tests.

\[
f_c = f'_c \left[ \frac{2 \cdot \varepsilon_c}{\varepsilon_o} - \left( \frac{\varepsilon_c}{\varepsilon_o} \right)^2 \right] \quad \text{or} \quad (3-3)
\]

\[
f_c = - \frac{f'_c}{\varepsilon_o^2} \cdot \varepsilon_c^2 + \frac{2 \cdot f''_c}{\varepsilon_o} \cdot \varepsilon_c \quad (3-4)
\]
The effect of increased load rates on the compressive strength of concrete has long been realized. Takeda and Tachikawa (1971) proposed a relationship between stress, strain, and strain rate from test results of 14 different batch mixes with varying aggregate sizes subjected to strain rates ranging from 1 $\mu\varepsilon/s$ to 1 $\varepsilon/s$ (Figure 3-4). Despite the different mix designs and concrete properties, all specimens produced geometric similarities in their stress-strain-strain rate relationships. Most researchers agree that the compressive strength of concrete increases as much as 15% at strain rates of 0.02 $\varepsilon/s$, although the increase in modulus is more moderate (Fu et al. 1991). Some signal matching algorithms for dynamic wave analyses recognize these rate effects and compensate by varying the concrete modulus throughout the length of the pile. However, the modulus assigned to each level is again assumed constant and Hooke’s Law used to determine the stress.

Strain values in a pile exhibiting either elastic behavior or large amounts of side shear can vary significantly throughout the length of the foundation (location) as well as throughout the loading event (time). During rapid and dynamic tests, these strain measurements are almost never in phase and therefore add a degree of difficulty when determining the ultimate capacity. Consider the strain at multiple levels throughout the drilled shaft in Figure 3-5. Not only is it evident that the upper level gages experience higher strains, but they undergo much larger strain rates than the lower level gages. Also prominent is a delay of maximum strain, or phase shift, at each level. Had the maximum strains occurred simultaneously at each gage level, then perhaps compensation for strain rate effects may not prove to be worthwhile when calculating the ultimate capacity, since
the strain rate at maximum strain is zero. However, the ultimate capacity in rapid and dynamic tests usually occurs at a point in time between the maximum strains of the upper and lower levels. Because of this, strain rate effects must be considered, and unloading stresses must be accurate.

The nonlinear hysteretic behavior of concrete coupled with its sensitivity to increased loading rates suggests that the use of a constant concrete modulus and a linear stress-strain relationship may be inappropriate for certain load tests that exhibit a wide range of strains and strain rates throughout a pile. Regardless of the type of load test used or analysis procedure performed, provisions for a variable modulus and/or nonlinear hysteretic stress-strain model may prove to refine the test results and ultimately yield a more accurate interpretation of the foundation performance. This forms the basis of this chapter.

3.3 Laboratory Study

In order to confirm the results of previous researchers and identify the strain rate effects indicative of rapid load tests, thirty 50 mm x 100 mm (2 in x 4 in) cylindrical mortar specimens were cast and tested at various loading rates using a MTS 809 Axial/Torsional Test System and a 180 kN (20 T) laboratory-scale rapid load testing device (Stokes, 2004). Due to the limitations of the rapid load testing device, smaller diameter mortar cylinders were chosen as test specimens to ensure that failure was achievable, especially with an anticipated rate-dependent strength increase of 15%. Though aggregate scaling issues existed between the mortar specimens and published
concrete data, results of Takeda and Tachikawa indicate that their behaviors should be similar.

Table 3-1 contains the mix design for the batch of specimens. Each specimen was instrumented with 2-10 mm resistive type strain gages located at mid-height and 180 degrees apart. Data was collected using a data acquisition device at various sampling rates appropriate for the loading rate of each specimen.

Table 3-1 Specimen Mix Design.

<table>
<thead>
<tr>
<th>Mix Details</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Batch Volume</td>
<td>0.014 m³</td>
</tr>
<tr>
<td>w/c ratio</td>
<td>0.485</td>
</tr>
<tr>
<td>Mini-slump pat diameter</td>
<td>63.61 cm²</td>
</tr>
</tbody>
</table>

3.3.1 Phase 1

The first 23 cylinders were tested to failure using the MTS compression device at load rates that ranged from 0.7 to 2788 kN/sec (Table 3-2). A minimum of two cylinders were tested at a given load rate as is customary with ASTM standards. Scaled testing caps were made to replicate those used in larger cylinder tests and reduce the frictional restraining forces between the ends of the specimens and the testing platens.
3.3.2 Phase 2

The remaining 7 cylinders were tested using the laboratory-scale rapid load testing device. This device was used to achieve representative rapid load test strain rates that were beyond the limitations of the MTS device. Though the intent was to load each cylinder to failure, 3 of the tests did not provide adequate force to break the cylinders, therefore unload data was inadvertently obtained. This mishap proved fortunate in the later developmental stages of the stress-strain model.

3.4 Laboratory Results and Model Formulation

Results from the first testing phase and the cylinder breaks from the second testing phase showed that the stress and modulus at any given strain increased with increasing load rate (Figure 3-6). Though the strength exhibited a distinct increase, any trend in the ultimate strain was indiscernible. If assumed that the strain rate remains constant throughout the test as is loading rate, then the individual tests can be plotted in a 3-dimensional space as seen in Figure 3-7. In order to determine the best-fit surface through

<table>
<thead>
<tr>
<th>Number of Specimens</th>
<th>Load Rate (kN/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>0.7</td>
</tr>
<tr>
<td>2</td>
<td>2.4</td>
</tr>
<tr>
<td>2</td>
<td>7.7</td>
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<tr>
<td>3</td>
<td>861.6</td>
</tr>
<tr>
<td>3</td>
<td>2788.1</td>
</tr>
</tbody>
</table>
these data points, the Hognestad parabolic stress-strain model was used to fit the two tests run at the upper ASTM loading rate limit (0.7 kN/sec). This well-accepted parabolic relationship proved to be a relatively simple but reliable model for fitting the ASTM load rate data (Figure 3-8).

To better understand the strain rate effects and to expand the Hognestad relationship into the strain rate dimension, all data falling within constant values of strain (strain bands) were plotted in the stress-strain rate plane (Figure 3-9). From this, a cubic relationship for each constant-strain band was determined and applied to the base Hognestad model such that a best-fit surface could be created through the data (Figure 3-10). A more familiar surface appears when the strain rate is converted to log-scale as is traditionally published (Figure 3-11).

This model was developed with the assumption that both the strain rate and load rate remained constant throughout each test. However, examination of the data as a function of time revealed that they varied significantly. Variations in the loading rate are largely due to the logic statements in the computer code and feedback controls attempting to hone the testing device on a specified loading rate. Although variations in the strain rate are partly a result of the load rate control, they are mostly the result of the nonlinear behavior of the mortar. As previously discussed, a simple linear stress-strain model does not adequately describe the nonlinear stress-strain response of concrete and/or mortar, therefore it should not be expected that the time derivatives be directly proportional.

\[ f_c \neq \varepsilon_c \therefore \dot{f}_c \neq \dot{\varepsilon}_c \]  \hspace{1cm} (3-5)
If an instantaneous strain rate is calculated for each test instead of assuming that the strain rate remains constant, a slightly different plot emerges (Figure 3-12). After performing the same procedure described above for determining the best-fit surface, a model was developed which exhibits a similar shape but more subtle increase in ultimate strength with increasing strain rate (Figure 13).

\[
f_c = -\frac{f'_c}{\varepsilon_o^2} \cdot \varepsilon_c^2 + \frac{f''_c}{\varepsilon_o^2} \cdot \varepsilon_c \cdot \left[ \frac{5}{4} \cdot \left( \frac{\dot{\varepsilon}_c - \dot{\varepsilon}_o}{\varepsilon_o} \right)^\frac{1}{3} + 2 \right]
\]  

(3-6)

Upon close examination, it can be seen that the above equation is a modified version of the published Hognestad relationship. The modification is in the form of a strain rate multiplier applied only to the “B” coefficient of the base parabolic equation. As a result of the data analysis, it was determined that modifications to the “A” coefficient led to no sizeable effects that could not be accounted for entirely in the “B” coefficient. A statistical analysis was performed against the raw data and their corresponding modeled values which yielded a coefficient of determination of 0.993. Again, the modeled surface displays a familiar geometric shape when the strain rate axis is plotted in log-scale (Figure 3-14).

Another attempt at developing a modeled surface was made; however, this model was based on a logarithmically increasing stress as a function of strain rate (Eqn. 3-7).

Though the logarithmic-based multiplier promised a better fit at lower strain rates, it did not represent the increase in stress beyond 0.2 \( e/sec \) well. Despite this downfall, the logarithmic model yielded a coefficient of determination of 0.992, and although both
models seemed to match well, the cubic model was chosen due to its consistency with published results.

\[
f_c = -\frac{f'_{c_2}}{\varepsilon_o^2} \cdot \varepsilon_o^2 + \frac{f'_{c_0}}{\varepsilon_o} \cdot \varepsilon_c \cdot \left[ \frac{1}{17} \ln \left( \frac{\dot{\varepsilon}_c}{\dot{\varepsilon}_o} \right) + 2 \right]
\]  \hspace{1cm} (3-7)

In an attempt to apply the model toward all concrete/mortar types, the base Hognestad model and the strain-rate compensating multiplier were normalized to all three input parameters: the ultimate strength (\( f'_{c_2} \)), the strain at ultimate strength (\( \varepsilon_o \)), and the strain rate at which the ultimate strength and strain were determined (\( \dot{\varepsilon}_o \)). All values can be determined from cylinder tests run in accordance with ASTM C39. Though it has been noted that the strain rate and load rate do not remain constant throughout a compression test, the ASTM standard makes provisions for this variability by stating that “the designated rate of movement shall be maintained at least during the latter half of the anticipated loading phase of the testing cycle.” For the purposes of normalization, the strain rate \( \dot{\varepsilon}_o \) is taken near the latter half of the testing cycle where it remains fairly constant.

\[
f_N = 2 \cdot \varepsilon_N \cdot \left[ \frac{1}{50} \cdot (\dot{\varepsilon}_N - 1)^{\frac{1}{3}} + 1 \right] - \varepsilon_N^2
\]  \hspace{1cm} (3-8)

When plotting a single test where failure occurred, the stress path can be seen as it maneuvers closely along the modeled surface (Figure 3-15). However, in a test specimen
where failure did not occur, the stress path deviates from the surface as the strain rate decreases (Figure 3-16). Once the strain rate decreases to within ASTM rates, the stress does not decrease as the model predicts, but remains higher. This led to the presumption that the model was only valid for specimens that were loaded to failure where the strain rate was in a continually increasing state. For tests that did not experience failure but underwent a load cycle, the model sufficiently predicts the stress up to the point of maximum strain rate; but afterwards, the measured stress follows a different path.

By plotting the test data from the Phase 2 specimens that underwent a load cycle, it was determined that beyond the point of maximum strain rate (hereafter referred to as the transition point), constant strain bands decreased linearly (Figure 3-17). A second surface was fit to model these data beyond the transition point (Figure 3-18). It is important to note that the origin for the linear decrease of the second model is entirely based on the stress-strain relationship defined at the location of the transition point. Depending on the magnitude of loading, the stress-strain relationship from diverges from a common path beyond the transition point (Figure 3-19). Prior to reaching the transition point, the A and B coefficients of the parabolic stress-strain curve are changing based on the cubic model (Eqn. 8). When the transition point is reached, the strain rate multiplier on the B coefficient remains constant for the remaining portion of the test, and a linearly decreasing offset is applied to the entire relationship (Eqn. 3-9). This offset is a function of strain-rate and was derived using only data in the decreasing strain-rate portion of the test. Despite a second inflection in strain rate near the end of the loading cycle (Figure 3-20), this linearly offset model sufficiently predicts the end-of-test stresses.
Figures 3-21 and 3-22 offer a three-dimensional outlook on the normalized stress path taken by one of the specimens (STN 6) and the modeled stress as determined by the pre-transitional cubic model and the post-transitional linear model (Figure 3-23). The coefficient of determination is 0.997, thereby showing that there is good agreement between the measured and modeled stress.

### 3.5 Case Studies

During the development stage of the model, input values were normalized with respect to the ultimate compressive strength, the strain at ultimate strength, and the strain rate at which these values were determined. The intent was to define a model that could be applied to the regression of concrete pile load test data despite the many possible variations in the concrete mix design. In order to test the viability of the model, two case studies were examined where stress, strain, and strain rate information were available. These case studies were selected due to the presence of modulus gages and concrete strength information. In both cases, the upper level strain gages were located such that there was no appreciable loss of load due to load shed. This ensured that the true axial stress at the gage location could be reasonably computed and provide a means for comparison.
3.5.1 Houston Shaft S-1

As part of a collaborative effort between the University of South Florida (USF) and the University of Houston (UH) to demonstrate the effectiveness of post grouting, four drilled shafts were constructed then tested using a 16 MN statnamic load testing device (Mullins and O’Neill 2003). Two of the 1.2 m (4 ft) diameter drilled shafts were embedded in sand, and the other two were embedded in clay. One shaft from each set served as a control (ungrouted), while the other was post grouted prior to load testing. For the purposes of this article, the particular shaft of interest is the ungrouted control embedded in sand (Shaft S-1).

Shaft S-1 was constructed to 6.4 m (21 ft) using 18-#9 reinforcing bars with #4 ties on 15 cm (6 in) centers. Strain gages were placed on the reinforcing cage so as to correspond to changes in the soil strata at 1.2 m (4 ft), 2.1 m (7 ft), and 6.1 m (20 ft). Each strain gage level consisted of four resistive-type electrical strain gages positioned 90 degrees apart.

Since the upper level strain gages were located close enough to the ground surface to reasonably assume no losses due to load shed, the measured statnamic force was used to determine the concrete stress at the upper level gage location. Because the force was being applied to a composite cross-section, a portion of the load had to be discounted due to the presence of the reinforcing steel. This was accomplished by using the measured strain, known cross-sectional area of steel, and Hooke’s Law to determine the force in the steel. Also, inertial effects were considered since the load cell and the strain gages were separated by a 3518 kg mass of concrete and steel. After these adjustments were made,
the corrected force was divided by the cross-sectional area of concrete to obtain the stress at the upper gage level.

Using the information from the concrete cylinder compression tests, the ultimate strength (26.2 MPa or 3.8 ksi), ultimate strain (0.001862 ε), and strain rate (40 µε/sec) were determined and the data normalized. Figure 3-24 shows the normalized data plotted with the modeled response. Good agreement was found between the data and the model despite the differences in the mix design between the concrete drilled shaft from which the data was taken and the mortar cylinders from which the model was developed (coefficient of determination of 0.980).

3.5.2 Bayou Chico Pier 15

The Bayou Chico bridge project involved the replacement of an existing drawbridge in Pensacola, Florida with a newer high-rise bridge. One of the 600 mm square prestressed piles located at Pier 15 of the new bridge became part of an extensive load test program implemented by the Florida Department of Transportation (FDOT) that compared cycles of static, statnamic, and dynamic load tests (Lewis 1999). Prior to casting, vibrating wire and resistive type strain gages were mounted 180 degrees apart at five levels throughout the pile: 0.71, 2.57, 4.55, 6.6, and 8.53 m from the toe. The pile was driven to a depth of 8.4 m which resulted in the uppermost gages being positioned above ground surface.

A dynamic test was recorded on the pile during the last blow of installation, after which three consecutive static load cycles were performed. Nearly two months later, a
A statnamic test was completed using a 14 MN device. For the purposes of this study, only results from the statnamic test are considered.

An inertial correction was applied to determine the stress at the upper level strain gages and similar calculations performed as outlined in the previous case study; however, no correction was made on behalf of the prestressing strands. Since the strands remain in tension throughout the load test, any compressive stresses imparted by the device are taken entirely by the cross-sectional area of concrete. The only compensation for prestressing was applied as an initial compressive offset at strand release of 7.6 MPa.

No concrete cylinder compression test information was readily available for this particular case study, therefore information from previous data regressions was used to determine the ultimate strength (49.3 MPa or 7.15 ksi). The ultimate strain (0.0027 ε) and strain rate (40 µε/sec) were assumed based on cylinder compression test results of similar piles. Figure 3-25 shows the normalized data plotted with the modeled response. Again, good agreement is notable between the data and the model (coefficient of determination of 0.979). When plotting the normalized results of both case studies, the difference in pile type is accentuated by the prestressing offset visible in the Bayou Chico pile (Figure 3-26).

### 3.6 Model Application

Since the results from the case studies indicate that the model provides a reasonably accurate prediction of upper level gage stresses in both the cast-insitu drilled shaft and the precast driven pile, the model was applied to a load test scenario to determine the effect on the regressed data. The scenario involves a relatively long drilled shaft
(length to diameter ratio of 49) tipped in soft soil but which exhibits a significant amount of side shear when rapidly loaded to geotechnical failure. These conditions highlight the need to address variations in strain magnitude and rate throughout a given test shaft.

The hypothetical shaft is 1.22 m (4 ft) in diameter and 61 m (200 ft) in length with strain gages located at ground surface and depths of 15 m (49 ft), 30 m (98 ft), 55 m (180 ft), and 60 m (197 ft). Longitudinal reinforcement is continuous throughout the length of the shaft such that the cross-sectional area of steel and concrete at all gage levels is 117 cm² (18 in²) and 1.156 m² (12.44 ft²) respectively. The concrete strength \( f_c' \), ultimate strain \( \varepsilon_o \), and strain rate at which the compression test was performed \( \dot{\varepsilon}_o \) is 27.58 MPa, 0.0027 \( \varepsilon \), and 40 \( \mu \varepsilon / \text{sec} \).

The measured load test data is presented in Figure 3-27. Variations between the strains at each gage level implies that a large portion of the applied load is taken by side shear. Also predominant is the characteristic phase lag between peak strain values seen in rapidly loaded piles. Both phenomena lend the data to a segmental analysis (Mullins et al. 2002). However, to perform the analysis, it is necessary to determine the force or stress applied at each gage level.

The stress at each gage location within the shaft is computed using three methods: (1) a linear elastic stress-strain relationship based on an ACI modulus, (2) a linear elastic stress-strain relationship based on a back-calculated modulus from the top gages, and (3) a nonlinear hysteretic stress-strain relationship based on the proposed model. The ACI equation produces a concrete modulus of 24.68 GPa (3580 ksi), that when weighed with a
steel modulus of 200 GPa (29000 ksi), yields a composite modulus of 26.44 GPa (3834 ksi). The calibrated composite modulus as determined by the ground surface gage level (back-calculated modulus) is 21 GPa (3046 ksi).

Once the stress/load at each gage level is computed, the difference in load between gage levels is divided by its corresponding segment surface area to obtain dynamic t-z curves which are uncorrected for inertia and damping effects (Figures 3-28 through 3-32). Segment 1, the portion of the shaft between gage level 1 and 2, shows a 21% reduction in maximum stress when the back-calculated modulus is used instead of the ACI modulus. All other segments, including the toe of the shaft, show the same reduction. This constant reduction in load is a direct result of a 21% reduction between the back-calculated and the ACI calculated composite moduli.

When considering the segment loading based on the modeled stress, a distinct shift in the distribution of load along the length of the shaft becomes noticeable. The upper level segments (1 and 2) do not distribute as much load as the back-calculated modulus curve suggests, whereas the lower level segments (3 and 4) distribute more. This occurrence can be explained using the segment stress-strain plots located within each figure (Figures 3-28 through 3-32).

When observing the stress-strain relationship of segment 1 (Figure 3-28), although barely distinguishable, the linear modulus gage relationship intersects the modeled relationship slightly above the point of maximum stress. As the magnitude of strain and strain rate decreases (moving downward through the shaft), the point of intersection begins to fall below the point of maximum stress on the model (Figures 3-29 through 3-32). This
is again a by-product of the nonlinearity of the concrete material and results in the overestimation of stresses in the upper segments and underestimation of stresses in the lower segments.

At this point, a segmental unloading point analysis (SUP) could be performed on the data to obtain the equivalent static response of each segment (see Lewis, 1999; Winters, 2002; and Mullins, et al., 2002). However, it is not the intent of this study to promote a particular analysis procedure or execute a complete analysis but to merely highlight the possible variations in deep foundation segmental load distribution as a result of modulus selection, or rather the selection of a particular stress-strain-strain rate relationship.
Figure 3-1 Instrumentation Scheme in a Long Pile.

Figure 3-2 Back-Calculated Elastic Modulus from Near-Surface or Modulus Gages.
Figure 3-3 Assumed Linear Stress-Strain Relationship vs. Measured Response.

Figure 3-4 Normalized Concrete Stress-Strain-Strain Rate Relationship (Adapted from Takeda and Tachikawa (1971)).
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4.0 Detection of Structural Failure from Rapid Load Test Data

Although rapid load testing is primarily intended to verify the geotechnical capacity of foundations, in certain instances it has shown structural deficiencies in drilled shafts that could possibly go undetected with conventional load testing. The mechanism of detection is afforded by the inherent features of a rapid load test. Therein, the measured force and the resulting measured displacement and acceleration traces can be used to identify an irregular response which is inconsistent with an intact shaft.

This chapter presents the background of the measurements as well as several case studies where the approach was unfortunately necessary.

4.1 Introduction

The ever-increasing load demands of larger structures require more attention to reliability issues that plague drilled shaft foundations. Therein, problematic regions consisting of voids and debris inclusions cast into the shaft during construction have undermined the integrity, and in some cases the load carrying capacity, ultimately resulting in an unuseable shaft. These anomalies can be attributed to poor construction techniques and numerous other factors such as small CSD ratios (minimum clear cage spacing to maximum aggregate diameter) (Garbin 2003; Deese 2004), low concrete slumps, sidewall sloughing, loose debris at the toe, high sand contents in the drilling fluid, and the disruption of setting concrete (Mullins and Ashmawy 2005). In order to
verify the soundness or integrity of the final product, it is common practice to use some form of non-destructive testing method.

Integrity testing includes a vast array of tests which can be generally categorized according to the physical principles upon which they are founded: visual, sonic, nuclear, and thermal. Visual inspections are usually performed by lowering a camera into the borehole prior to concrete placement to inspect the cleanliness of the borehole. Though this method seems relatively simple, it can only identify the presence of debris and offers no indication of the existence of an anomaly after construction is finished. Sonic testing, however, is the most widely used method and can lend insight into the general size and location of an anomaly. Sonic testing encapsulates a broad range of tests that are fundamentally based on wave propagation mechanics through cured concrete. In one form of sonic testing, Cross-hole Sonic Logging (CSL), a transmitting and receiving probe are raised simultaneously through adjacent tubes cast within the shaft (Olson Engineering, Inc. 2004). Sonic pulses are transmitted through the concrete between the tubes and wave arrival times recorded. Deviations from an anticipated wave arrival time are used to interpret the uniformity of the concrete matrix and identify aberrant conditions. In a nuclear test, specifically Gamma-Gamma Testing (GGT), a probe is lowered through an access tube similar to that used in CSL (Mullins and Ashmawy 2005). Radiation is emitted into the surrounding concrete and the reflected photons measured to determine localized density changes corresponding to voids or inclusions within an approximate 100 mm (4 in) radius (Mullins et al. 2003). Thermal Integrity Testing (TIT) is a new technology which measures the heat of hydration throughout a
shaft prior to curing. Similar to CSL and GGT, a thermal probe is passed through an access tube cast within the shaft. Temperature readings are taken along the length of the shaft in four directions for each access tube. Voids or soil inclusions are identified by “cold spots” where heat is not generated due to the lack of cementitious material. Each of these integrity methods are useful in identifying potential deficiencies; however, certainty can not be established without some form of destructive testing (e.g. coring or excavation).

Like integrity testing, load tests are often used as a means of quality assurance to determine if the as-constructed pile will carry the required design loads. In rare occasions, certain load tests can offer additional insight into the integrity of the pile. This article purports to highlight the additional benefit of rapid load tests in their ability to distinguish between geotechnical shear failure of the shaft/soil interface and structural failure (generically referred to hereafter as failure) as a result of deficiencies cast within the shaft.

4.2 Rapid Load Tests as an Integrity Test

Though load tests are intended to be nondestructive in nature, deficiencies cast within a shaft during construction can cause failure. In rapid load tests, these failures can often be misconceived as a by-product of the violent nature of the test. However, the influence of higher loading rates on the strength of concrete has long been realized. In loading rates typical of rapid tests (200 to 500 MN/s for statnamic), concrete compressive strength can increase as much as 15 percent (MacGregor 1997). In light of this, it is
difficult to charge the test type with the “breaking” of a pile. In reality, a static test of the same pile would have broken at a lesser load. In most cases, the destruction is due to an inadequate shaft cross-section but can be detected using information gathered during the test.

Load tests can be generally categorized according to their load duration, where rapid load tests (ASTM 7120) are shorter in duration than static tests (ASTM 1143) but longer in duration than dynamic tests (ASTM 4945). In each test, there exists a minimum amount of instrumentation necessary to capture the loading event and the foundation response. Typically, displacement transducers and/or accelerometers are used to ascertain the foundation settlement via direct measurements or numerical integration respectively. Both types of transducers have a range of applicability which dictates their usage and ultimately makes each transducer inherent to a particular load test. Displacement transducers are ideal for measuring pile response during the long load durations characteristic of static tests, while accelerometers are more suitable for the short durations characteristic of rapid and/or dynamic events. Though both transducers can ultimately offer a measure of the foundation settlement, only the acceleration trace recorded during a rapid or dynamic test can offer definitive insight into the occurrence of a failure. Depending on the type of load test and instrumentation used, simple mathematics can be used to determine and compute the location of a structural failure. A case study is presented below where the procedure was unfortunately necessary.
4.3 Florencia

The Florencia condominiums is a multi-story, drilled shaft-supported structure located in downtown St. Petersburg, Florida. In September 1998, two of the 0.914 m (36 in) diameter production drilled shafts were each rapidly load tested with a single cycle from a 14 MN (1600 T) statnamic device. The target load of 10.7 MN (1200 T) was 65% of the laboratory reported concrete strength for that particular day (25.5 MPa or 3.7 ksi). However, excavation of the upper 5 to 7 feet of both shafts exposed a concrete failure that occurred during the test (Figures 4-1 and 4-2). Both shafts exhibited the conical shape typical of a concrete compression failure. The following section describes the analytical approach used to decipher the data from one of the two tests (Shaft 84) and determine the existence, location, and actual concrete strength of the failure region.

The first indication of an aberrance can be found in the recorded load pulse and acceleration trace. This qualitative analysis consists of an examination of the geometric shape of the curves but requires a basic familiarity with the load test mechanics and typical results. During an initial investigation, the measured statnamic load pulse exhibits a drastic reduction following maximum load (Figure 4-3). This uncharacteristic reduction indicates that something has gone amiss during the test. Upon closer examination, the first deviation from a typical load pulse occurs between points 1 and 2. This deviation is further pronounced by a sharp discontinuity in the acceleration trace (Figure 4-4).

In the regression of rapid load test data, it is common to simplify the shaft/soil system as a single degree of freedom system consisting of a spring and dashpot. The
equation describing the loading event can then be written as:

\[ F_{\text{applied}} = kx + ma + cv \] (4-1)

In this equation, it is seen that the applied/measured force is composed of an equivalent spring/static force \((kx)\), an inertial force, \((ma)\), and a damping force \((cv)\). Since the applied force and acceleration are measured and the shaft mass is generally known, the applied force is usually inertia-corrected \((F_{\text{applied}} - ma)\) such that an analysis can be performed to determine the two unknown parameters, the spring stiffness \((k)\) and the damping coefficient \((c)\). In keeping with this tradition, if the mass of Shaft 84 (35974 kg) is assumed constant and the inertia-corrected load computed, the load-displacement response shows an abnormal decrease when compared to a typical trend (Figure 4-5). This abnormally low region in the inertia-corrected force compounded with the discontinuities in acceleration serve as indicators that a constant mass assumption is not valid.

In a forensic interpretation of the data, it appears that the original concrete failure occurs between points 1 and 2 where the acceleration originally deviates from the norm. Beyond point 2 the acceleration exhibits erratic behavior which can be interpreted as a brief form of resistance followed by more concrete crushing. A more quantitative analysis can be performed to identify the location of the aberrance with the use of some fundamental physics.

Eqn. 4-1 can be rearranged to show that the inertial force at any point in time is equal to the measured force and losses due to external soil resistance \((kx)\) and system damping \((cv)\).
\[ F_{\text{applied}} - kx - cv = ma \]  \hspace{1cm} (4-2)

If assumed that no significant change in inertia occurs between points 1 and 2, then the inverse ratio of the accelerations can be used to compute the ratio of the masses (Serway, 1996).

\[ \frac{m_1}{m_2} \equiv \frac{a_2}{a_1} \]  \hspace{1cm} (4-3)

However, application of this equality hinges on the assumption that between points 1 and 2, the individual components on the left side of Eqn. 4-2 vary such that the net external force on the system remains constant (left side of the equation remains constant). It is evident in Figure 4-3 that a slight decrease in the measured force occurs at the onset of concrete crushing, but changes in the equivalent spring and damping force are not as readily apparent.

As the deficient region begins to fail, the length of shaft upon which load is applied effectively reduces to the upper segment that is bound by the point of load application and the point of concrete failure. This drastic reduction in the effective length significantly alters the total shaft mass, stiffness, and damping field such that the terms \( m, k, \) and \( c \) decrease. Had the failure occurred instantaneously, it may have been sufficient to assume continuity in the displacement and velocity. However, the failure occurs over a relatively lengthy amount of time, so both displacement and velocity increase. For this reason, it is difficult to accurately conclude whether the sum of net external forces remains constant throughout failure. Therefore, it is assumed herein that
no appreciable difference exists, and the failure between points 1 and 2 is considered to occur instantaneously.

If the mass balance from Eqn. 4-3 is applied to the data between points 1 and 2 where the accelerations are -17.5 and -215 m/s² (-1.78 and -21.9 g’s ) respectively, the mass ratio is 0.08. Multiplying the ratio by the original length of the shaft (24 m or 79 ft) results in the effective length of the shaft after failure (1.9 m or 6.3 ft) and thereby gives an approximate location below the point of load application for the deficient region. In regards to the actual strength of the concrete at the onset of failure, assuming no losses to load shed, the concrete failed at 6.7 MN (757 T) or 10.3 MPa (1.5 ksi). If considering the load rate effects on concrete, the strength can be reduced to 9 MPa (1.3 ksi). This value corresponds to a 35% strength reduction from the laboratory reported results.

In a report to the foundation contractor, the testing agency stated that the failure was “a result of insufficient concrete strength at the time of testing,” and the authors emphasized that they did not “have enough data to determine the reasons why the concrete did not have sufficient strength.” In any case, the monitoring of shaft acceleration inherent to the rapid test proved beneficial in identifying the concrete deficiency and alarming the engineers. Had a static load test been performed, then acceleration information would not have been available, and the load response could have easily been misinterpreted as a geotechnically weak shaft (Figure 4-6).
Figure 4-1 Broken Shaft After Load Testing.

Figure 4-2 Top 1.5 to 2.1 m (5 to 7 ft) of Broken Shaft After Excavation.
Figure 4-3 Florencia Shaft 84 Load Pulse vs. Typical Results.

Figure 4-4 Florencia Shaft 84 Acceleration Trace vs. Typical Results.
Figure 4-5 Florencia Shaft 84 Inertia-Corrected Load vs. Typical Results.

Figure 4-6 Possible Failure Modes from Load Test Results.
5.0 Conclusions and Recommendations

Since load tests results are generally considered as the basis of performance from which foundations can be designed, it is imperative that the analyzed load test data be as accurate as possible. However, uncertainty still surrounds some of the parameters necessary for determining the foundation resistance. Similar to other fields of engineering, reasonable assumptions are made to simplify the analysis and arrive at an answer that may not be entirely correct but is typically regarded as acceptable. The following sections highlight some general conclusions drawn from an investigation of some common assumptions.

5.1 Statnamic Damping Coefficient

The current, most widely-accepted method for analyzing statnamic data is the Unloading Point Method (UPM), or some variation thereof. There are two assumptions upon which the method is based: the static capacity of the pile is constant while plunging and the damping coefficient is constant throughout the test. The damping coefficient is calculated over a region near the end of the test; therefore, the calculated value of the damping coefficient is only valid over this region. When a pile exhibits a purely elastic behavior, the associated damping coefficient is valid over the entire test. However, when a pile surpasses elastic behavior and begins yielding, the calculated damping coefficient is no longer valid within the elastic region but only within the yielding region. Since
application of the UPM, previous case studies have noted that the derived damping coefficient:

• is valid over much of the load cycle for piles that exhibit purely elastic behavior,
• provides a good estimate of the ultimate static capacity when yielding is observed, but
• does not serve as a good estimate of the static capacity within the elastic region of piles which exhibit yielding.

The intent of this study was to entertain the hypothesis that damping is more closely associated with the actively straining soil not necessarily constant for a given foundation/soil system. Though the numerical model was not created to predict specific numerical values, the computer-generated data reinforces the results of previous case studies by offering a plot of the statnamic damping coefficient which was back-calculated from modeled static and statnamic tests on shafts of identical in situ conditions. The plot of this "true" statnamic damping coefficient shows values that are initially higher prior to yielding which then asymptotically degrades through a transitional stage, where they are met and coincide with the UPM-calculated damping coefficient.

The similarity in shape of the UPM-calculated damping coefficient and the displacement-dependant volume change (herein referred to as the DDVC) strongly suggest that such a relationship exists. This is further reinforced by the similarity in shape with the back-calculated damping coefficient, at least at small strains (prior to and just after shear failure).
Further modeling, compounded with full-scale load tests, could provide an arsenal of data with which to establish a quantifiable relationship between the DDVC and the statnamic damping coefficient. If estimations of the statnamic damping coefficient can eventually be established through correlations with the DDVC, then perhaps an analysis procedure can be adopted that will provide the reliability of the UPM at ultimate capacity as well as an accurate prediction of static capacity at small displacements.

5.2 Concrete Stress Determination

Long piles subjected to significant amounts of side shear are often constructed with embedded strain gages positioned at strategic locations throughout the pile so as to better distinguish the load contributing components (end bearing vs. side shear) and develop the side shear load distribution. In load tests that produce low concrete strains and/or strain rates, an approximated linear stress-strain relationship and constant modulus may prove to be sufficient for determining the stress at strain gage locations. Tests exhibiting low strain rates but high strains are best evaluated using a non-linear, parabolic stress strain relationship. However, in tests that produce both high strains and strain rates, the evaluation of the true concrete stress requires more sophisticated analysis. The presented approach incorporates the effects of strain magnitude, strain rate, and the rate of change of strain rate. As a result, the concept of a linear elastic modulus in concrete is virtually unusable.

Based on the results of the nonlinear hysteretic model developed herein, certain conclusions can be drawn:
• A linear stress-strain relationship (constant modulus) at best slightly over predicts stresses at upper gage levels and under predicts stresses at lower gage levels; in the worst case it misrepresents all gage levels.

• Since the use of ACI and modulus gage values are constant modulus assumptions, the segment t-z curves will show similar geometric shapes, but differ in value by the same degree as the difference in assumed moduli. But, neither accounts for the true nonlinearity or strain rate dependency.

• Since most foundations are not loaded to structural failure, a concrete stress-strain-strain rate relationship purely based on concrete break data is inadequate in describing load/unload cycles found in load tests.

• In all concrete specimens that undergo a load cycle, the stress follows a common surface up to the point of strain inflection or maximum strain rate (herein referred to as the transition point). However, each specimen returns along a different path defined by the strain rate at the transition point which can vary depending on the rate and magnitude of loading.

In order to implement the proposed model toward the regression of load test data, the following procedures are recommended:

• Concrete cylinder compression tests should be performed in accordance with ASTM C39 on specimens from each pile to identify the ultimate compressive strength \( f'_c \), the strain at ultimate strength \( \varepsilon'_o \), and the strain rate at which the ultimate strength and strain were determined \( \dot{\varepsilon}_o \). If analyzing static load test data, the Hognestad formula with no modification for strain rate is sufficient.
Strain gages should be placed at or near ground surface such that the measured load can be used to define the concrete stress-strain relationship in a region where there is no load shed. This measured load should be inertia-corrected and discounted depending on the reinforcement. Although it is ideal to place the gages above the ground surface, some piles and shafts do not extend above the ground. In these circumstances, the gages should be placed reasonably shallow.

Using the values from the concrete cylinder compression tests, normalize the stress, strain, and strain rate (if necessary) at the upper gage level. Compare the normalized data against the modeled response to determine whether the model sufficiently predicts the stress path.

If the model sufficiently predicts the stress at the upper gage level, then apply the model to all gage levels.

If performing a rapid load test, further regress the data to obtain the equivalent static response.

5.3 Detection of Structural Failure

Drilled shaft foundations have been historically plagued with integrity issues which oftentimes result in an unusable or questionable shaft. Numerous integrity tests exist which identify potentially deficient regions within the concrete shaft, and though most of these tests are non-destructive in nature, certainty can not be established without some form of destructive investigation.
Load tests are generally used as a means of verifying that a pile has sufficient capacity to withstand required design loads and provide performance-based acceptance. These tests can either fully define the soil/foundation interactions or merely prove the capacity to a desired level (e.g. 1.5 to 2.1 times the design load). Under no circumstances is the foundation expected to fail structurally. However, voids or inclusions cast within a shaft can cause structural failure if the cross-section cannot sufficiently withstand the applied stresses. Due to the load shed characteristics of a pile, it is more probable that these structural failures will occur in upper portions where the applied stresses are higher. In most cases, determination of the failure mode may prove difficult, short of coring or excavating. However, certain load tests may provide additional information regarding the integrity or soundness of a pile via certain instrumentation.

Rapid load tests inherently utilize instrumentation which provides the ability to distinguish between the geotechnical shear failure and structural concrete failure of a pile. It is specifically the measured acceleration which constitutes the mechanism for the detection. In a test where erratic responses suggest structural failure, a simple mass balance can be performed to identify the proximity of the suspected failure. By definition, static load tests lack the rate-dependent effects that when monitored can be used to verify the integrity of the shaft and distinguish between structural and geotechnical failures. Therefore, structural failures that occur during a static load test may be misinterpreted as a geotechnically weak shaft. This misinterpretation could prove detrimental to a structure if the test pile is truly representative of the production piles.
As most nondestructive test methods can verify the presence of a structural failure after occurrence, in many cases they are not conducted. To this end, rapid and/or dynamic load testing provides both load carrying evaluation and integrity verification. In any event, post static load test integrity evaluation may be prudent.
References


About the Author

Michael Jeffrey Stokes, the second son of James D. Stokes and Sue Ann Graves was born in Tampa, Florida on October 25, 1976. After high school, he joined the United States Army and was stationed in Ft. Drum, New York. After a 4-year initial enlistment, Michael returned home where he continued his military service in the Florida National Guard and attended college at Hillsborough Community College. In 2000, he transferred to the University of South Florida where he received a Bachelor’s, Master’s, and Doctorate of Philosophy in Civil Engineering. On June 18, 2005, Michael married Amy Wacaser, the sister of his best friend, and they currently reside near his childhood home in Plant City, Florida.