DEVELOPMENT OF MODIFIED T-Z CURVES FOR LARGE DIAMETER PILES/DRILLED SHAFTS IN LIMESTONE FOR FB-PIER

By

LILA DHAR NIRAULA

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This document is dedicated to my parents, my wife and my son.
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THE PRIMARY PURPOSE OF THIS RESEARCH WAS TO DEVELOP NEW P-Y SPRINGS IN ORDER TO ACCOUNT FOR THE MOMENT TRANSFER DUE TO AXIAL SIDE SHEAR BY PERFORMING TESTS ON SCALED MODELS OF DRILLED SHAFTS IN A GEOTECHNICAL CENTRIFUGE. THE STUDY WAS DONE IN ROCK FOR THE LARGEST SIDE SHEAR EFFECTS AND SYNTHETIC ROCK MADE OUT OF CRUSHED LIMESTONE, CEMENT AND WATER WAS USED TO REPRESENT FLORIDA LIMESTONE. THE LATTER HAS BEEN SHOWN TO HAVE
remarkable similarities with natural Florida limestone in terms of its strength and stiffness characteristics.

Data obtained from 7 axial tests and 12 lateral tests were used to develop P-Y curves for rock. Currently, P-Y curves do not exist for rock in FB-Pier software and rock is modeled as clay with rock strength and stiffness properties which leads to very conservative designs of piles/drilled shafts in rock.

The results of axial and lateral tests reveal that the contribution to the moment transfer through side shear may be significant, especially for large diameter piles/shafts socketted in high strength rock. The study also shows that the current models of computing P-Y curves for rock are very conservative and implementation of the new P-Y curves is recommended in FB-Pier software to model the correct transfer of lateral load from pile/shaft to rock.
CHAPTER 1
INTRODUCTION

1.1 Background

Before 1980, most Florida Department of Transportation (FDOT) bridges were founded on small diameter pre-stressed concrete piles. However, in the past 20 years, the use of drilled shaft foundations as an alternative to driven piles for supporting bridges and buildings has become increasingly popular. The main reasons for their use are fivefold: 1) presence of shallow limestone, 2) the need to resist large lateral loads such as wind loads (hurricanes, tornados etc.) and ship impacts, 3) right of way constraints which require minimal foundation footprints, 4) need to minimize construction noise and vibrations in urban areas and 5) the economy of replacing large number of piles with a single or few drilled shafts without pile caps. With the introduction of larger and more autonomous equipment, the shaft diameters have been getting larger and larger. Shaft diameters in excess of 8 ft are now common in Florida.

1.2 Current Drilled Shaft Design in Florida Limestone

Currently, the FDOT design of bridge pier foundations for extreme events from lateral loading (e.g., ship impact) is done with the finite element code FB-Pier. In FB-Pier, pile or drilled shaft’s soil-structure interaction is characterized with non-linear T-Z and P-Y springs. The vertical non-linear springs (T-Z) transfer the axial pile/shaft loads to the soil/rock while the horizontal non-linear springs (P-Y) transfer the pile/shaft shear loads. In addition, the P-Y and T-Z springs are attached to the pile/shafts’ beam element at the element’s centerline, as shown in Figure 1.1.
In the field, the actual transfer of axial load from the pile/shaft to the soil/rock takes place at the pile/shaft and soil/rock interface and not at the centerline of the pile/shaft as shown in Figure 1.2. Because of the shear transfer at a distance, a moment or couple ($\tau$-$D$, Figure 1.2) develops. This moment may be significant (especially in strong material or in the case of large diameter pile/shafts) in addition to the P-Y contribution.
1.3 Modified FB-Pier Model

In order to represent the moment contribution due to the axial shear (Figure 1.2), it is proposed that an additional rotational spring be attached at the pile/shaft centerline as shown in Figure 1.3. The moment resistance due to the shear transferred at the walls $M_s$ is a function of shaft diameter, rock/soil strength (secant stiffness of the T-Z curve), and the angle of rotation ($\theta$) of the pile/shaft. The rotational secant stiffness, $K_{\theta\theta}$, of the spring (Figure 1.3) is $M_s/\theta$. Note that the new rotational stiffness at the shaft/pile nodes requires no new material characterization (or parameters).

![Proposed FB-Pier Model](image)

Figure 1.3 Proposed FB-Pier Model

1.4 Validation of Proposed FB-Pier Model

In order to validate the new model, both axial and lateral load test data are necessary. Due to limited availability of this data from the field, it was decided that both lateral and axial tests be carried out in the centrifuge. In the centrifuge, the field loading and stress conditions can easily be reproduced and the soil/rock parameters can easily be controlled. It was also proposed that the tests be carried out on rock since the axial moments will be large due to the strength of rock vs. soil, as well the fact that little if any T-Z and P-Y information is available for Florida Limestone. Consequently, in addition to
developing rotational springs (Ms-$\theta$), testing Florida limestone will allow development of representative P-Y curves for rock.

Florida limestone has highly variable strength in both the horizontal and vertical directions. Moreover, there are over ten specific formations in Florida (Tampa, Ocala, etc.). In addition, since the experiments will have both lateral and axial loading carried out on the same rock (i.e., strength, density etc.), as well as repeated several times (i.e., verification), it was believed that field samples could not be employed. Consequently, it was decided to use synthetic rock to perform the centrifuge tests. The latter has been shown to be homogeneous, isotropic, with properties similar to those of the natural Florida limestone (Cepero, 2002). Synthetic rock with the required strength may be made by mixing ground up limestone, cement and water in various proportions.

1.6 Scope of Work

In order to develop both the P-Y curves, as well as validate the Rotational Spring Model (Figure1.3), three different strengths of rock were tested- weak, medium and strong to cover the range of Florida limestone. Based on the previous research (Kim, 2001) as well as FB-Pier modeling, it was decided to vary the strengths from 10 tsf to 40 tsf. For lateral loading, two rock strengths, $q_u$, were considered, 10tsf and 20tsf, whereas for axial, three were tested, 10tsf, 20tsf and 40tsf. The highest strength was not tested for lateral because of the small expected lateral movements in the rock, and the large variation over small vertical distance (top one diameter of shafts). The lateral test results were extrapolated for strengths below 10 tsf and above 20 tsf.

For each strength of rock tested, the following were varied:

- Loading (Axial and Lateral): Axial tests were needed to validate the existing T-Z curves as a function of rock strengths. Lateral tests were required to back compute
P-Y curves, as well as estimate the axial moment-rotation (rotational springs) curves along a shaft’s length.

- Shaft Diameter: Two different diameters were tested to investigate the effect of diameter on the rotational springs and P-Y springs. Diameters of 6ft and 9ft were used, as they are representative of the typical large diameter shafts in Florida.

- Length to diameter (L/D) ratio: For each shaft diameter 2 L/D ratios were considered to investigate the effect of embedment depth. L/D ratios for different rock strengths and shaft diameters were chosen based on FB-Pier modeling (described in Chapter 2) and field experience (Kim, 2000).

Table 1.2 below shows the nineteen centrifuge tests which were used to calculate T-Z and P-Y curves, as well as validate rotational spring model. A number of other tests (axial and lateral), were performed, but the data in Table 1.2 was repeatable and subsequently used in this research.

Table 1.1 Centrifuge Tests Performed

<table>
<thead>
<tr>
<th>Rock q&lt;sub&gt;u&lt;/sub&gt; (tsf)</th>
<th>Shaft Dia (ft)</th>
<th>Shaft Length (ft)</th>
<th>L/D ratio</th>
<th>Loading Type</th>
<th>No of Tests</th>
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<td>10</td>
<td>6</td>
<td>18</td>
<td>3</td>
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<td>3</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>18</td>
<td>3</td>
<td>lateral</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>24</td>
<td>4</td>
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<td>9</td>
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<td>40</td>
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<td>18</td>
<td>3</td>
<td>axial</td>
<td>2</td>
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CHAPTER 2
CENTRIFUGE MODELING AND REPRESENTATION OF FLORIDA LIMESTONE
WITH SYNTHETIC ROCK

2.1 Centrifuge Background

The UF centrifuge used in this study was constructed in 1987 as part of a project to study the load-deformation response of axially loaded piles and pile groups in sand (Gill 1988). Throughout the years several modifications have been undertaken to increase the payload capacity of the centrifuge. Currently, electrical access to the centrifuge is provided by four 24-channel electrical slip-rings and the pneumatic and hydraulic access is provided by a three port hydraulic rotary union. The rotating-arm payload on the centrifuge is balanced by fixed counterweights that are placed prior to spinning the centrifuge. Aluminum C channels carry, i.e., support both the pay-load and counterweights in the centrifuge.

On the pay-load side (Figure 2.1), the aluminum C channels support the swing-up platform, through shear pins. The latter allows the model container to rotate as the centrifugal force increases with increasing revolution speed (i.e., rpm). The platform (constructed from A36 steel), and connecting shear pins were load tested with a hydraulic jack in the centrifuge. The test concluded that both the swing up platform and shear pins were safe against yielding if the overall pay-load was less than 12.5 tons (Molnit, 1995).
Figure 2.1 The UF Geotechnical Centrifuge

2.1.1 Theory of Similitude

Laboratory modeling of prototype structures has seen a number of advances over the decades. Of interest are those, which reduce the cost of field-testing as well as reduce the time of testing. Additionally, for geotechnical engineering, the modeling of insitu stresses is extremely important due to soils’ stress dependent nature (stiffness and strength). One way to reproduce the latter accurately in the laboratory is with a centrifuge.
A centrifuge generates a centrifugal force, or acceleration based on the angular velocity that a body is traveling at. Specifically, when a body rotates about a fixed axis each particle travels in a circular path. The angular velocity, $\omega$, is defined as $d\theta/dt$, where $\theta$ is the angular position, and $t$ is time. From this definition it can be implied that every point on the body will have the same angular velocity. The period $T$ is the time for one revolution, and the frequency $f$ is the number of revolutions per second (rev/sec). The relation between period and frequency is $f = 1/T$. In one revolution the body rotates $2\pi$ radians or

$$\omega = 2\pi / T = 2\pi f$$  \hspace{1cm} \text{Eq. 2.1}$$

The linear speed of a particle (i.e., $v = ds/dt$) is related to the angular velocity, $\omega$, by the relationship $\omega = d\theta/dt = (ds/dt)(1/r)$ or

$$v = \omega r$$  \hspace{1cm} \text{Eq. 2.2}$$

An important characteristic of centrifuge testing can be deduced from Equations 2.1 and 2.2: all particles have the same angular velocity, and their speed increase linearly with distance from the axis of rotation ($r$). Moreover, the centrifugal force applied to a sample is a function of the revolutions per minute (rpm) and the distance from the center of rotation. In a centrifuge, the angle between the gravitational forces, pulling the sample towards the center of the earth, and outward centrifugal force is 90 degrees. As the revolutions per minute increase so does the centrifugal force. When the centrifugal force is much larger than the gravitational force the normal gravity can be neglected. At this point the model will in essence feel only the “gravitational” pull in the direction of the centrifugal force. The earth’s gravitational pull ($g$) is then replaced by the centrifugal pull ($a_c$) with the following relationship;
Centrifugal acceleration: \[ a_c = r \left( \frac{\pi \text{rpm}}{30} \right)^2 \]  
Eq. 2.3

where \[ \text{rpm} = \frac{30}{\pi} \sqrt{\frac{a_c}{r}} \]  
Eq. 2.4

Scaling factor: \[ N = \frac{a}{g} \]  
Eq. 2.5

\[ N = \sqrt{\frac{a_c^2 + g^2}{g^2}} \]  
Eq. 2.6

If \( a_c >> g \), \[ N = \frac{a_c}{g} \]  
Eq. 2.7

where
\( a = \) the total acceleration
\( g = \) the normal gravitational acceleration
\( a_c = \) the centrifugal acceleration
\( \text{rpm} = \) number of revolutions per minute
\( r = \) distance from center of rotation.

The scaling relationship between the centrifuge model and the prototype can be expressed as a function of the scaling factor, \( N \) (Equation 2.5). It is desirable to test a model that is as large as possible in the centrifuge, to minimize sources of error (boundary effects, etc.), as well as grain size effects with the soil. With the latter in mind, and requiring the characterizing of foundation elements with 18 to 27 ft of embedment in the field, the following rationale was employed to determine the appropriate centrifuge \( g \) level and angular speed, \( \omega \).

The height of the sample container was 12 inches and its diameter was 17 inches. The longest foundation to be modeled (27 ft embedment) if tested at 67 gravities would
require a model depth of 4.84 inches, which would ensure that the bottom of the foundation model had seven inches of rock beneath, minimizing end effects. Spinning the centrifuge at lower gravities would imply the model would have to be larger making it impossible to conduct more than one test in the rock cast in the container.

Knowing that the desired scaling factor $N$, was 67 gravities, and that the distance from the sample center of mass to the centrifuge’s center of rotation was 1.3 meters (51.18 inches), it is possible to compute the angular speed of the centrifuge, $\omega$ from Equation 2.1,

$$\omega = \frac{30 \sqrt{67 \times 9.81 m/s^2}}{1.3m} = 215 \text{ rev/min} = 3.58 \text{ rev/sec} \quad \text{Eq. 2.8}$$

The actual scaling factor, $N$, from Equation 2.6 is:

$$N = \sqrt{(67 \times 9.81)^2 + 9.81^2} \div 9.81 = 67.01 \quad \text{Eq. 2.9}$$

Based on Equation 2.5, a number of important model (centrifuge) to prototype (field) scaling relationships have been developed (Bradley et al, 1984). Shown in Table 2.1 are those, which apply to this research.

Based on Table 2.1, two of significant scaling relationships emerge:

- Linear Dimension are scaled 1/$N$ (prototype length = $N \times$ model length)
- Stresses are scaled 1:1.

The first significantly decreases the size of the experiment, which reduces both the cost and time required to run a test. The second point, ensure that the insitu field stresses are replicated one to one. Note that the effective stress controls both the stiffness and strength of the soil and rock.
Table 2.1 Centrifuge Scaling Relationships (Bradley et al., 1984)

<table>
<thead>
<tr>
<th>Property</th>
<th>Prototype</th>
<th>Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Acceleration (L/T^2)</td>
<td>1</td>
<td>N</td>
</tr>
<tr>
<td>Dynamic Time (T)</td>
<td>1</td>
<td>1/N</td>
</tr>
<tr>
<td>Linear Dimensions (L)</td>
<td>1</td>
<td>1/N</td>
</tr>
<tr>
<td>Area (L^2)</td>
<td>1</td>
<td>1/N^2</td>
</tr>
<tr>
<td>Volume (L^3)</td>
<td>1</td>
<td>1/N^3</td>
</tr>
<tr>
<td>Mass (M)</td>
<td>1</td>
<td>1/N^3</td>
</tr>
<tr>
<td>Force (ML/T^2)</td>
<td>1</td>
<td>1/N^2</td>
</tr>
<tr>
<td>Unit Weight (M/L^2T^2)</td>
<td>1</td>
<td>N</td>
</tr>
<tr>
<td>Density (M/L^3)</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Stress (M/LT^2)</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Strain (L/L)</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Moment (ML^2/T^2)</td>
<td>1</td>
<td>1/N^3</td>
</tr>
</tbody>
</table>

2.1.2 Slip Rings and Rotary Union

A total of 96 channels are available in the centrifuge through four slip rings (24 channels per slip ring) mounted on the central shaft (Figure 2.2). Each channel may be accessed from the top platform above the centrifuge, and used to obtain readings from instrumentation being used to monitor the model, or the centrifuge itself.

For this particular research, several channels were used to send voltages (power-in), and obtain readings (signal-out) from a 10000-lb load cell, two linear variable differential transducers (LVDT’s to measure deformation), twelve strain gage circuits (multiplexed into one channel) and one camera. Power was also supplied, through slip rings to solenoids, which controlled air supply to the air pistons (point load source, etc), and to an amplifier, which boosted the signal (load cell, etc) coming out. To minimize noise, cross talk, etc., low voltage out devices were kept on different sets of slip rings than the higher
voltage power input. For instance, the voltage-in for the load cell was 10 volts, however, the signal (voltage-out) coming from the instrument ranged from 0 to 20 milivolts.

Figure 2.2 Slip Rings, Rotary Union, and Connection Board (left)

The pneumatic port on the hydraulic rotary union was used to send air pressure to the air piston acting on the model. The air-line was then connected on the centrifuge through a set of solenoids (Figure 2.3), located close to the center of rotation. Solenoids have the advantage that they may be operated independently of each other, allowing the application of air pressure to a large number of pistons in any combination required. The solenoids required an input voltage of 24 volts of direct current and opened or closed values depending if voltage was supplied or not.
2.2 Synthetic Rock

It was readily recognized from the variability of Florida limestone, and the
difficulty of cutting and placing rock cores within the centrifuge container (i.e., fit), that
an alternative approach was needed. To replicate the stress-strain and strength
characteristic of limestone, it was decided to use a combination of ground up limestone,
cement and water, i.e., synthetic rock.

Synthetic limestone (Gatorock) was developed by mixing ground up limestone
sieved through a No 10 standard sieve (maximum particle diameter of 2mm), Portland
cement and water. The proportions of these constituents were varied to achieve different
strengths. An extensive program of preparation and testing was carried out with different
amounts of constituents in order to achieve 3 strengths ranging from about 10 to 40 tsf.
Once the desired strength was achieved the test was repeated 3 to 5 times to quantify repeatability. It was found that when the water content was at least 20% by weight of the cement aggregate mix, the strengths were repeatable. Unconfined compressive strengths of the final test samples varied by 10% only. However, specimens with less than 20% water had greatly increased strength variability. This strength variability is most probably due to improper mixing of the constituents at lower water content. Table 2.3 shows 14-day unconfined compressive strengths and Figure 2.4 shows rock samples ready for testing.

2.2.1 Raw Materials

Limestone Products Inc. provided the crushed limestone used in “Gatorock” from their quarry in Newberry, Florida, west of Gainesville. The quarry mines limestone from the Ocala formation, which outcrops in this region. FDOT Research and Materials Office, Gainesville transported it in bulk to UF.

Figure 2.4 Synthetic Rock Samples
The crushed limestone contained oversized particles, which had to be removed in order to obtain uniform strength and consistency. This was done by first drying the material in oven for at least 24 hours and sieving it through a No 10 standard sieve. Material retained on the No 10 sieve was removed for a well graded particle distribution (see Figure 2.5).

2.2.2 Mixing and Curing

As part of any mix, cylindrical samples (2in diameter 4in high) were cast by hand. First the required weight of the aggregate (dry, sieved limestone) was weighed. The percentage of cement (Quickrete type I cement) was based on the weight of the dry aggregate and the percentage of water was based on the sum of the weights of aggregate and cement. For example, suppose that samples containing 10% cement and 20% water are to be made. If the desired weight of aggregate for a batch of samples is 4 lb then the weight of cement used was 0.4 lb (10% of 4lb) and the weight of water was 0.88 lb (20% of 4.4lb). The mixture was compacted into brass molds (2inch diameter and 4 inch high) and cured at room temperature for 14 days before testing for its unconfined compressive strength.

The synthetic rock samples used for the axial and lateral load testing in the centrifuge were much bigger (1.8 ft³) and it required use of a concrete mixer for its preparation. A drum mixer manufactured by Constructional Machinery Co, Iowa, with a rotational speed of 19 rpm was used for this purpose. This mixer with the capacity of 3.5 ft³ allowed the specimen to be prepared in a single batch. The sample containers used for testing were steel cylinders measuring 17 inches diameter and 12 inches high. Limestone aggregate and cement were mixed dry in the mixer and water was gradually added so that
the mixture formed was uniform. The mixture was then emptied into the sample container and compacted using a vibratory probe.

![Grain size Distribution for the Limestone Aggregates used in the Synthetic Rock.](image)

**2.3 Testing of Synthetic Rock Samples**

Two kinds of samples of synthetic rock were tested during this project to determine their unconfined compression and splitting tensile strengths. First, small trial mixes were made and cast in cylindrical samples (2 inches diameter by 4 inches high) and tested for their unconfined compressive strength. The testing was done using Humboldt loading frame located at UF geotechnical laboratory, used for tri-axial testing of soil samples. If the mix was satisfactory then large batches were made for the centrifuge bucket. For the latter batches, small cylinders (2” by 4”) were also cast for latter testing. The small cylinders were tested in unconfined loading generally on the day of the centrifuge test.
In addition, cores were taken from the centrifuge containers after the completion of axial and lateral testing of the model drill shafts. The small cores were tested in both unconfined compression and splitting tensile loading conditions.

Figure 2.6 shows the set up of the samples for unconfined compression and splitting tensile tests. The unconfined compression tests were performed in accordance with ASTM D 2938, which specifies the specimen L/D ratio of 2 to 2.5 and a rate of loading to achieve failure in 2 to 15 minutes. The specimen L/D ratio of 2 and a time to failure of about 5 minutes fell well within the ranges specified by ASTM. Splitting tensile tests were conducted following ASTM D 3967. The specimen L/D ratio of approximately 0.5 and the rate of loading to achieve failure in about 5 minute were again well within the ranges of 0.2-0.75 and 1-10 minutes respectively, specified by ASTM.

Figure 2.6 Synthetic Rock Strength Testing, (a) Unconfined Compression Test (b) Splitting Tensile Test
2.4 FB-Pier Analysis of Proposed Drilled Shafts Experiments

In order to determine the rock strengths for testing, properties of concrete for the shaft as well as the capacity of the load cell needed to measure the axial and lateral loads on the shaft, a FB-Pier analysis was performed. The analysis also gave the maximum depths that the moments and the shear forces would be transferred, determining shaft lengths, as well as instrumentation placement. In the analysis, the drilled shaft was modeled as a single shaft and tested for the maximum lateral as well as axial load before the rock failed. In the present FB-Pier software the axial behavior in rock is modeled by its own T-Z curves but the lateral P-Y characterization has to be modeled as either soft clay or stiff clay with rock properties. Analysis was done using both models. In general it was found that the soft clay model gave much higher capacities. Using a 9ft diameter shaft with rock strengths varying from 10 tsf to 45 tsf it was found that the maximum lateral load (required to fail 45 tsf rock) was approximately 6000 kips. This translates to a force of 1.3 kips in the centrifuge model spinning at a centrifugal acceleration of 67g. The axial capacity of the shaft was tested with and without tip resistance. With the tip resistance included it was seen that the maximum axial capacity of the shaft would be over 40000 kips (9 kips in the model). Ignoring tip resistance (by modeling the tip with very low strength clay) it was found that the maximum axial capacity of the shaft in skin friction would be about 17500 kips which corresponds to a force of 4 kips in the model. For the latter representation, placing a layer of styro-foam at the bottom of the shaft could eliminate the tip resistance. Hence it was decided that a load cell of 10 kips capacity would be sufficient for both types of tests. Both axial and lateral loading would be provided through a Bimba air cylinder with a 2500 lb capacity.
The analysis also gave the depth below which no moment and shear force would be transferred from the shaft to the rock (tip cut-off depth). It was found that this depth depended on the strength of the rock as well as the diameter of the shaft. In general it was observed that moments and shears could be transferred deeper in weaker rock and with larger diameter shaft. Table 2.2 summarizes the results of the FB-Pier analysis of the drilled shafts.

Table 2.2 Results of FB-Pier Modeling

<table>
<thead>
<tr>
<th>Rock $q_u$ (tsf)</th>
<th>Max Lateral Load (kips)</th>
<th>Max Axial Load (kips)</th>
<th>Tip Cut-Off Depth (Diameters)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>9ft (Prototype)</td>
<td>9ft (Model)</td>
<td>6ft dia shaft</td>
</tr>
<tr>
<td>10</td>
<td>3000</td>
<td>0.7</td>
<td>3500</td>
</tr>
<tr>
<td>25</td>
<td>5000</td>
<td>1.1</td>
<td>10000</td>
</tr>
<tr>
<td>35</td>
<td>5500</td>
<td>1.2</td>
<td>13000</td>
</tr>
<tr>
<td>45</td>
<td>6000</td>
<td>1.3</td>
<td>17500</td>
</tr>
</tbody>
</table>

The analysis suggests that for shaft diameters ranging from 6 to 9ft the cut-off depths should vary from 3 to 5 diameters. Hence, the L/D ratio should be kept less than 5 and varied depending on the rock strength and the shaft diameter, giving the longest possible shaft length of 45 ft. This could be modeled by an 8 inch shaft at an acceleration of 67g. The analysis also suggested that the maximum depth to which moments and shear forces are transmitted would not exceed 3 diameters even for rock with unconfined strengths higher than 40 tsf. It was therefore decided to conduct tests on rock strengths of 10 and 20 tsf. It was decided to use the result obtained from the tests to extrapolate to lower and higher strength rock.
2.5 Reduction of $q_u$ and $q_t$ Tests

For unconfined compression loading, the axial load, $P$ and vertical deformation, $\Delta$ are recorded. To obtain $q_u$, the $P$-$\Delta$ data has to be converted into stress ($q$) and strain ($\varepsilon$) results. The following equations were used to calculate $q_u$ and $\varepsilon$ from $P$ and $\Delta$.

\[
q_u = \frac{P}{A} \quad \text{Eq. 2.10}
\]
\[
\varepsilon = \frac{\Delta}{L} \quad \text{Eq. 2.11}
\]

where $A$ is the average initial cross sectional area and $L$ is the initial length of the sample, and $\Delta$ is the sample deformation.

The splitting tensile tests were reduced in accordance with ASTM D 3967 using the following equation.

\[
q_t = \frac{2P}{\pi L d} \quad \text{Eq. 2.12}
\]

where $P =$ maximum applied load, $L =$ sample length and $d =$ sample diameter.

Shown in Table 2.4 are the average results from the unconfined compression and split tensile tests for various mix designs. Each testing batch consisted of 3 samples and was repeated three to five times for repeatability.

Table 2.3 Percentage of Constituents for Different Strength Synthetic Rock

<table>
<thead>
<tr>
<th>Strength</th>
<th>$q_u$ (tsf)</th>
<th>$q_t$ (tsf)</th>
<th>Limestone</th>
<th>Cement</th>
<th>Water</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td></td>
<td></td>
<td>77.5</td>
<td>2.5</td>
<td>20</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>1.6</td>
<td>75</td>
<td>5</td>
<td>20</td>
</tr>
<tr>
<td>20</td>
<td></td>
<td>3.4</td>
<td>72.5</td>
<td>7.5</td>
<td>20</td>
</tr>
<tr>
<td>30</td>
<td></td>
<td>5.6</td>
<td>60</td>
<td>15</td>
<td>25</td>
</tr>
<tr>
<td>40</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
2.6 Model Shaft Design and Construction

2.6.1 Concrete

Since rock strength evaluated in the tests was to be as high as 20 tsf, the concrete strength had to be sufficient enough to ensure rock failure instead of the concrete. FB-Pier modeling (section 2.4) was performed with 5 ksi concrete (360 tsf). The latter are typical strengths reported in design plans for drilled shafts in Florida. Because of the small size of the model shaft (9 feet diameter prototype = 1.62 in diameter modeled in the centrifugal acceleration at 67g), it was not possible to use large aggregates. In addition, the curing time had to be limited because the shaft had to be ready for testing in about 4 days of casting in the rock. Leaving the concrete in the shaft to cure longer would result in increased strength of the synthetic rock.

Initially, many different types of ready mixes were tried but the highest strength reached was only 3000 psi. It was evident that the target strength of 5000psi was not viable from ready mixes. Next, silica sand and ordinary Portland cement mixes were tried but the highest strengths achieved were approximately 4000 psi. Subsequently, granite sand and Portland cement mixes were found to give strengths higher than 4000 psi. Adding silica fume to the granite sand cement mix and reducing the water to cement ratio through synthetic super-plasticizer resulted in compressive strengths of over 5000 psi. Note, the addition of silica fume and reduction of water not only increased strength but also reduced undesirable shrinking of the concrete.

2.6.2 Steel Reinforcement

It was decided that two different prototype (field) shaft diameters would be tested – 6ft and 9ft (model diameters of 1.08in and 1.62 in). For the model shafts, 0.75 in and 1.3 diameter steel pipes were used above the rock surface, and below ground, the pipes were
slotted to provide steel ratios ($\rho$) of 6.4% and 7.5%. The latter reinforcement was necessary to carry the expected moments that would transverse the rock interface. Winding a 0.0625-inch tie wire round the pipe provided shear reinforcement. The concrete cover for the reinforcing steel in both cases was approximately 0.1 inch, which is equal to 6.7 inches in the prototype shafts. Figure 2.7 shows the drilled shaft steel reinforcement along with attached strain gauges for both 6ft and 9 ft diameter shafts. The gauges were attached in pairs on opposite sides of the reinforcement. Figure 2.8 is a magnified view of the rebar cage for the 9 ft diameter shaft which shows the instrumentation in more detail.

Figure 2.7 Instrumented Rebar Cages for 6ft and 9ft Model Shafts
Figure 2.8 Model Rebar Cage for the 9ft Diameter Drilled Shaft Showing Instrumentation Details
CHAPTER 3
TEST EQUIPMENT AND PROCEDURE

3.1 Sample Container

For this research, the original sample container built by Molnit (1995) from aluminum alloy 6061 with rectangular dimensions of 18 inches long, 10 inches wide and 12 inches deep was to be used. It was the largest container available at the start of the research. Since other research was using the container, as well the need for long curing times (rock, shaft, etc.), it was recognized that the synthetic rock could not be cast directly in the container. Both the synthetic rock and the model-drilled shaft were prepared external to the container and subsequently placed in it for testing. However, preliminary testing of shafts constructed this way failed. Due to small unavoidable gap between the container and the sample, a crack (Figure 3.1), developed across the rock mass, in the direction perpendicular to loading. Although the gap between the container wall and the rock was filled with fine sand and compacted, this did not prevent cracking in the rock mass. The cracking was attributed to loss of confinement, which would not exist in the field.

It was hence decided to build new sample containers for this project. The new containers (6) were constructed from 0.5 inch thick steel pipes with inside diameter of 17 inches and overall length of 12 inches as shown in Figure 3.2. The hooks or ears were used in lifting the new containers with the cast insitu rock.
The new sample containers offered three distinct advantages over the old sample container. First, as the synthetic rock was cast directly into an individual container and not removed prior to testing, the container provided sufficient rock confinement which eliminated rock cracking during lateral loading. Secondly, the larger area of rock in the new containers made it possible to conduct multiple tests within the same rock sample. Finally, construction of more than one container allowed samples of rock to cure while others were being set up for centrifuge testing, i.e., staggered construction and testing.

### 3.2 Loading Frame and the Base Plate

Since the old rectangular sample container was replaced with a new cylindrical shape, the arrangement for loading and displacement measurement had to be modified. It was thought that instead of attaching the loading air piston directly to the container, a loading frame independent of the container would be a better arrangement to minimize boundary effects. In addition, the single frame should be capable of providing loading for both axial and lateral scenarios.

The frame was constructed out of square aluminum tube 2in by 2in in cross section and reinforced with 0.5 in diameter steel rods to prevent buckling during loading. The
frame was bolted to a 21in by 21in aluminum base on which the sample container would be attached. The base plate was 0.5 in thick and reinforced on the underside with aluminum angles in order to prevent buckling. Figure 3.2 shows the loading frame together with the new sample container and the base plate.

Figure 3.2 New Cylindrical Sample Container with Aluminum Loading Frame and Base Plate

3.3 Strain Gages and the Bridge Completion Circuits

3.3.1 Strain Gages

In order to monitor the moments, shear and rock lateral resistance along each shaft, multiple pairs of strain gages were attached to opposite sides of the shaft. To obtain moments, shears, etc., along the shaft, 6 pairs or sets of strain gages were attached at 6 elevations along the shaft. For accurate strain measurements, the gages were attached to the steel reinforcement in each shaft. The strain in the reinforcement was subsequently
translated to the strain on the outside surface of the shaft, then to rotations and finally to
moments. The rock lateral resistance (P-Y) was obtained by double differentiating the
moment distribution along the shaft at a given elevation.

Because of the small size of the model shafts, the strain gages had to be small in
size. However, strain gages smaller than 3mm are very difficult to install. Consequently,
a compromise was found (lab testing) using a gage length of 3mm. After an extensive
search of suitable strain gages it was decided to use CEA-06-125UN manufactured by
Measurements Group, Inc. A gage length of 1/8 in and a narrow geometry together with
large solder pads for connecting wires made this strain gage ideal for this research. The
gages have a gage factor (GF) of 2.1 with an excitation voltage of 10 volts. A gage
resistance of 350 ohms was selected vs. 120 ohms, giving a higher output voltage from
the bridge, which is more sensitive to strain changes.

3.3.2 Bridge Completion Circuits

Each strain gage was connected to its own separate quarter bridge circuit. Since
each gauge generates its own strain, then separate computation of axial, shear and
moment values at a given cross-section is possible.

Lead wire effect (change in resistance of the lead wires due to heating, stretching
etc) is a source of error in quarter bridge circuits. Using a three-wire quarter bridge circuit
(Figure 3.3), instead of two wires will reduce this effect. In a two-wire circuit, both lead
wires are in series with the strain gage, in one arm of the Wheatstone bridge. In the three-
wire circuit, the first lead wire remains in series with the strain gage but the second lead
wire is in series with the dummy resistor R₄ between the negative input and the output
corners of the bridge. If these two wires are the same type, length and exposed to same
temperature and stretching, their resistances will be the same and the bridge balance will be unaltered by any change in the two wires.

Figure 3.3 Three-wire Quarter Bridge Circuit for Conditioning Strain Gage Signals

3.4 Load and Displacement Measuring Devices

A load cell with a 10000 lb capacity was used to measure the applied load during either axial or lateral testing. Load was provided through a “Bimba” air cylinder. The air cylinder–load cell assembly was mounted on the loading frame either horizontally or vertically for either lateral or axial loading.

Two LVDTs with 0.5 inch of travel were used to measure the displacements of the top of the shaft for lateral loading. The LVDTs were attached to the top of the centrifuge sample container (Figure 3.2). The devices were fixed 1 inch apart, and measured lateral displacements at two separate points above the surface of the rock. Figure 3.4 shows the devices used for both loading and monitoring of force and displacement on each shaft.
3.5 Data Acquisition System

The outputs from the 12 strain gages, one load cell and two LVDTs were collected by a National Instrument data acquisition card (NI 6034E) and processed and monitored by LabView. Since the raw output voltage from the strain gage bridges were very small (i.e., milivolts), they had to be amplified prior to passing through centrifuge slip rings to increase their signal to noise ratio. Amplification was done by an Omega EXP-20 amplifier/multiplexer (Figure 3.5), which could amplify the signals 800 times.
Figure 3.5 EXP-20 with Instrumented Shaft and Bridge Completion Circuits.

The signals from the strain gage bridges were connected to 12 different input terminals of the EXP-20 and the amplified output was multiplexed into a single channel of the NI 6034 card in the computer. The output from the load cell (20 mV max) was also amplified before the signal was sent through the centrifuge slip rings. This was done using an Omega DMD-465 bridge sensor and amplifier which amplified the signal 1000 times. The signal from the LVDT (40V/in) did not need amplification and was connected to the NI6034 directly. Figure 3.6 shows the overall schematic of the data acquisition system, amplifiers, and instrumentation.

The LabView Virtual interface (VI), which converts the load cell and LVDT voltages into force and displacement, is shown in Figure 3.7. The amplified output voltages from the 12 strain gauges and load cell were stored by the same VI in a
Microsoft EXCEL worksheet file. The EXCEL worksheets have VB interface to convert the voltages into strains and loads. LabView also displays a graph of load vs. displacement of the top of the shaft to control the “Bimba” air cylinder during the test.

![Data Acquisition System Diagram]

**Figure 3.6 Data Acquisition System**

### 3.6 Test Procedures

#### 3.6.1 Preparation of the Model Shaft

Step involved in preparation of the model shaft are described below:

**3.6.1.1 Strain gaging**

Before casting the shaft in rock, the rebar cage for the model was instrumented with strain gages along its length. The steel pipe (with slots cut in it) representing the rebar cage was first ground flush with abrasive paper and then cleaned with acetone. This removed the grease, dust etc. from the bonding area to present the placement of the strain gages. Next, lines were marked with a scriber on the cleaned steel where the strain gages were to be bonded. The strain gages were then bonded to the steel surface using Cyanoacrylate strain gage adhesive and thumb pressure for approximately one minute to cure.
3.6.1.2 Wiring strain gages

Next, lead wires were soldered to the exposed strain gage solder tabs to connect them to their respective bridge circuits. Each strain gage has two terminals (solder tabs) for wiring. One wire was attached to the first terminal and two wires to the second so that the gage could be connected to a 3-wire quarter bridge circuit, Figure 3.3. A 25 pin connector provided the connection from the strain gages on the model shaft to the bridge completion circuit located on the centrifuge arm.
3.6.1.3 Coating strain gages

Since the shaft rebar cage and strain gages were to be embedded in concrete, the strain gages had to be protected well against moisture and other chemicals in the concrete. A variety of coating substances are available depending on the nature of the environment in which the gages are used. In the field, strain gages and lead wires are protected against such harsh conditions by many thick layers of different materials. However, the small size of the model did not allow thick multiple layers to be used and it was necessary to use a single thin layer. In the early tests, a single component: xylene coating (M-coat A) supplied by Vishay Measurements group was used but it was soon found to be ineffective. It was observed that moisture and other chemicals from the wet concrete could easily penetrate the coating and soften the Cyanoacrylate adhesive between the steel and the gages. For instance, the results from several early tests had to be discarded as the strains did not go high enough, possibly because of de-bonding of the gages from the steel of the rebar cages.

Other materials were tried to determine the most suitable coating to protect the strain gages and it was finally decided to use a two component epoxy coating (M-Bond AE-10) from the same supplier. This worked well as it is highly resistant to moisture and most other chemicals. No de-bonding of strain gages was detected on shafts with this coating.

3.6.1.4 Model shaft installation

The synthetic rock was prepared as described in chapter 2. After curing the rock for about 10 days, the model drilled shaft was installed in the rock mass (see Figure 3.8). The process is described as follows:
A hole of required diameter (1.08in for 6ft prototype and 1.62in for 9ft prototype) was drilled at the desired location on the surface of the synthetic rock sample. Initially, this was done using a “Hilti” hand drill. However, it was discovered that the diameter of the hole constructed this way was not uniform and decreased with depth. This was attributed to the difficulty of keeping the drill perfectly vertical while drilling. Subsequently, it was decided to use a drill press to create the shaft hole. This greatly improved the quality of the hole, although there was still some variation in diameter due to wobbling of the drill bit and shaking of the rock sample during drilling. Typically, the difference in diameter between the top and bottom of a 3 inch deep hole was 0.01 inches which is equal to 0.67 inches in the prototype hole which is typical.

Before placing the instrumented rebar cage in the hole, it was coated with a thin layer of bentonite slurry. The bentonite slurry was prepared by adding 3% by weight of dry bentonite powder in water and mixed using an electric mixer. The slurry was poured into the hole and kept there for few minutes before pumping it out. This formed a thin slurry cake on the wall.

In the actual drilled shaft construction, bentonite slurry is used to maintain the wall stability of the drilled hole in caving soils. The purpose of the slurry here was to provide a separating layer between the shaft concrete and the rock. Without this separating layer, the concrete in the model shaft was forming a chemical bond with the synthetic rock. This resulted in extremely high axial and lateral capacity of the model shaft as the synthetic rock and the shaft concrete were behaving as a single material. In practice, the bond between the shaft concrete and rock is of a mechanical nature, with no chemical
bonding. The bentonite layer was kept very thin so that its effect on the axial shear capacity of the shaft/rock interface was minimal.

Next, the instrumented rebar cage was lowered into the hole with the strain gages aligned in the direction of loading. Concrete (prepared as described in chapter 2) was poured into the hole and compacted with a small steel rod by layers. The rebar cage was kept at the middle of the hole in the concreting process with a wood template placed on the top of the rock surface. The shaft was then left to cure and gain strength for 4-5 days before testing began in the centrifuge.

Figure 3.8 Installation of the Model Shafts in the Synthetic Rock

3.6.2 Centrifuge Test Setup

When the model shaft was ready for testing, the sample container was bolted to the base plate and the loading frame attached to it (Figure 3.2). The Bimba pneumatic cylinder with a 10000 lb capacity load cell was then mounted to the loading frame. Depending on the type of test (axial or lateral) the air cylinder was placed either vertically or horizontally with a stainless steel rod extending from the load cell to the
model shaft (Figure 3.9). In the case of lateral tests, the loading occurred at 0.8 to 1 inch above the rock surface. It was desirable to load the shaft as close to the rock surface as possible in order to avoid large moments developing on the shaft above the rock and to transfer high moments into the rock. The size of the load cell allowed the minimum distance to be about 0.8 inches. For the axial tests, the load-measurement assembly was mounted on a horizontal member (vertical LVDT attached), which applied axial load at the top of the shaft about 3 inches above the rock surface.

![Figure 3.9 Test Set Up for Axial and Lateral Loading in the Centrifuge](image)

Two LVDTs monitored the displacements in both the lateral and vertical loaded shaft tests. In the axial set up, the LVDTs were attached to a horizontal bar clamped to the top of the model shaft (Figure 3.9), with the tips of the LVDTs resting directly on the rock surface. For the lateral set up, the LVDTs were placed horizontally with their tips pushing against the shaft about 1 inch apart. They were mounted on a horizontal bar, which was clamped to two points on the edge of the sample container. It was decided to place the LVDTs as close to the rock surface as possible to compare with integrated strain displacements. Because of size and space constraints, the distance from the surface to the lower of the two LVDTs was 0.5 inches. In both loading scenarios, the system of
displacement measurement was independent of the loading frame. The latter was
important, since the frame was expected to deflect under loading.

3.6.3 Centrifuge Balance Calculations

Before spinning, the balance condition of the centrifuge arms was checked. This
involved comparing the sum of moments from the payload vs. sum of moments on the
counter-weight side about the centre of rotation of the arm. The moments on each side
were calculated by multiplying the weight of each component by the distance from its
centroid to the center of rotation of the arm. The difference in the total moments between
the two sides is called the out of balance moment and the value had to be less than one
percent of total applied moment. To correct any unbalance, additional weights were either
added or removed from either the payload (experiment) or counterweight side of the
centrifuge. For each rock sample, the total weight of the sample container was slightly
different (340–345 lbs) which made it necessary for the additional weight to be added or
removed for each test. A spreadsheet was used to tabulate weights and distances from the
centre of rotation for each major component of the centrifuge.

3.6.4 Centrifuge Testing

The assembled sample container was lowered unto the centrifuge sample platform
with the aid of a crane. The base plate was fixed to the platform with four 3/4 inch steel
bolts. The air line was connected to the Bimba air cylinder. The LVDTs, load cell and
the strain gages were connected to their respective plugs which provided both power as
well as monitored output signals from the devices with the LabView software.

The testing consisted of spinning the centrifuge at 215 revolutions per minute
(rpm) giving a centrifugal acceleration of 67g at the rock surface (1.3m from the axis of
rotation) and loading (axially or laterally) the shaft to failure while recording the load,
displacements and the strain data. In both types of test, failure was indicated by a large increase in displacement with very small or no increase in load.

Before spinning the centrifuge, all electrical connections were checked carefully, any loose wires were attached to the centrifuge arm with duct tape and the floor was cleaned to remove any loose material. All the electrical outputs (12 strain gages, LVDTs and the load cell) were measured with a digital voltmeter and verified with the corresponding readings shown by LabView in the computer. This was especially important for the strain gage bridge and the load cell outputs as they were small (few millivolts) and had to be amplified before sending them to the computer. This also ensured that the amplifiers were functioning properly.

The centrifuge was started and accelerated to 215 rpm. When all the readings (strain gage bridges, LVDTs, load cell) were stable, loading was initiated by slowly applying pressure to the air cylinder by turning the control knob located in the control room. The air pressure was increased in steps of about 1 psi until the model shaft failed. As the shaft was loaded, load, deflection and the strain gage outputs were observed on the computer monitor.

LabView plotted a graph of load versus deflection of the shaft during the test. As described earlier, LabView also stored an EXCEL worksheet of raw voltages from the strain gages, LVDTs and load cell. Reduction of data to obtain T-Z and P-Y curves is described in Chapter 4.
4.1 Axial Load Tests

Instrumented axial load tests provide both axial force and displacement along a shaft as a function of applied load. Of interest is the side shear, \( \tau \) vs. axial displacement (z), i.e., T-Z curves. The T-Z curves are important in the development of the rotational springs (Chapter 1), as well as moment equilibrium at a cross-section.

Initially, it was planned to instrument the model-drilled shaft with six pairs of strain gages as in the case of lateral loading (Figure 4.2). From each pairs of strain gages, a compressive strain for each applied axial load state is found. Subsequently, each strain may be converted into stress (\( \sigma \)) or compressive force (Q). The difference between the compressive forces between adjacent pairs of strain gages is the force (T) transferred from the shaft to the rock. Dividing this force by the area of the shaft between the two sets of strain gage locations gives the unit skin friction (\( \tau \)) at the middle of the shaded element (Figure 4.2). The axial displacement (z) of the shaft at that location is obtained by subtracting the sum of all the strains to that location from the axial displacement of the top of the shaft. Hence, graphs of \( \tau \) vs. z (T-Z curves) could be plotted at five elevations along the shaft.

However, since the synthetic rock was uniform with depth, the skin friction was not expected to vary with depth. Further, the use of styro-foam at the bottom of the shaft eliminated end bearing. Hence, the T-Z curve was obtained without the use of any strain gages (see Figure 4.1) as follows:
40

Figure 4.1 Axial Loading with no Strain Gages

- Using the applied load on top of the shaft (P) and zero axial force at the bottom (Styrofoam – 0 end bearing) the total load transferred to the rock (T) is equal to the applied load (P).

- The shear stress on the shaft wall (skin friction, $f_s$) is then given by

$$f_s = \frac{T}{\pi DL} \quad \text{Eq. 4.1}$$

where, D= shaft diameter and L = length of shaft embedded in rock.

- A plot of $f_s$ against the measured axial displacement ($z$) of the shaft is the T-Z curve.

In the method described above, axial compression of the shaft has been ignored as it was found to be negligible.

4.2 Lateral Load Tests

The goal of the lateral load tests was to obtain P-Y curves for rock at various unconfined strengths. Two types of P-Y curves were back computed; 1) Current FB-Pier
small diameter pile/shaft curves which are not corrected for shaft friction effects, and 2) Proposed FB-Pier model, P-Y curves corrected for the side shear. The theory for the back computation of the latter curves is presented in 4.2.3. In order to calculate, the P-Y curves, the moment at a given cross-section must first be computed from the strain gage data.

The model shaft was instrumented with 6 pairs of strain gages as shown in Figure 4.2. The raw data from each lateral load test consist of load and displacement at the top of the model drilled shaft and the 12 strain gage bridge outputs (voltages) along its length. The reduction of raw data to obtain P-Y curves involves numerous steps as described bellow.

![Figure 4.2 Strain Gage Layout for Lateral Tests](image)

Figure 4.2 Strain Gage Layout for Lateral Tests
4.2.1 Strain from Bridge Output

The strain for a single strain gage quarter bridge (Figure 3.4) can be calculated using the following equation,

$$\varepsilon = \frac{4\Delta V}{GE}$$  \hspace{1cm} \text{Eq. 4.2}

where,

- $\varepsilon$ = strain
- $\Delta V$ = change in bridge output voltage
- $G$ = gage factor
- $E$ = bridge excitation voltage.

This equation assumes a linear variation of strain with change in strain gage resistance and is obtained from the following relationship,

$$\varepsilon = \frac{\Delta L}{L} = \frac{\Delta R}{RG}$$  \hspace{1cm} \text{Eq. 4.3}

where,

- $R$ = unstrained strain gage resistance
- $\Delta R$ = change in resistance due to strain,
- $G$ = gage factor.

4.2.2 Determination of Bending Moments from Strains

Calculation of bending moments from strains involved the following steps:

4.2.2.1 Fitting trend lines to raw strain data

After calculating strains for all 12-strain gages, the strains were plotted against the lateral load on the top of the shaft, as shown in Figure 4.3. The strains from the gages on the opposite sides of the shaft are plotted. The positive strains are tension and the negative strains are compression. Plots similar to Figure 4.3 were computed for all six
locations along a shaft, (Figure 4.2). Appendix A plots all the strain vs. load for the 12 lateral tests (Table 1.1, Chapter 1) conducted.

Evident from Figure 4.3, the strain–lateral load relation is linear initially and then becomes highly non-linear. Analysis with FB-pier and observations (Figure 4.3), suggest the strains become non-linear at about 130 micro-strains. The latter corresponds to the initiation of concrete cracking. Cracking results in the movement of the shaft’s neutral axis resulting in non-symmetrical strains assuming that plane sections remain plane.

![Figure 4.3 Strains from a Pair of Gauges vs. Applied Lateral Load on a Model Shaft](image)

The plot of lateral load vs. strain was used two ways. First, the plot identifies any irregularities in the strains. For instance, non-continuous strain profiles (i.e., jump discontinuities) are strong indicator of either broken or slipping strain gages. Second, from the plot of strain vs. load, a trend line or relationship may be established to compute moments for any lateral load of interest. Also, the output voltage contained noise due to
centrifuge slip rings, wiring, etc. The trend lines were subsequently used in computing the bending moments. Figure 4.3 shows the trend-lines together with their equations (generally 5th order) for each strain gage.

4.2.2.2 Calculation of bending strains

At each strain gage location, the total strain consists of an axial and bending component. Of interest is the bending strain, \( \varepsilon_b \) at any given section of the shaft,

\[
\varepsilon_b = \frac{\varepsilon_1 - \varepsilon_2}{2}
\]

where, \( \varepsilon_1 \) and \( \varepsilon_2 \) are the values of the strain on the opposite sides of the shaft.

The following diagram (Figure 4.4) shows in detail how the bending strains are obtained from the measured strains.

![Figure 4.4 Total, Axial and Bending Strains on Cross-Section](image-url)
4.2.2.3 Development of moment-curvature relation

For small bending strains, the moments can be calculated as follows

\[ M = EI \frac{e_b}{r} = EI\phi \]  

Eq. 4.5

where, \(E = \) Young modulus, \(I =\) second moment of area, \(r = \) shaft’s radius and \(\phi\) is the curvature of the cross-section given by \(\phi = \frac{e_b}{r}\).

As identified earlier, the maximum strain for which relationship in Equation 4.5, applies (i.e., constant EI) is strains up to 130 micro-strains, Figure 4.3. Above that strain level, the stress-strain behavior of concrete is nonlinear (E not constant), and the concrete cracks in tension, shifting the neutral axis (I not constant). Therefore, a non-linear Moment (M) and curvature (\(\phi\)) relationship is needed to calculate the bending moment. The relationship still has the form \(M = B\phi\), where the curvature, \(\phi\), is still given by \(\frac{e_b}{r}\), but EI is no longer constant and must be replaced by the variable B. The non-linear M- \(\phi\) relation was developed in 3 different ways and compared to ensure the appropriate bending moment diagram along the shaft in order to obtain the P-Y curves.

For a shaft of given cross-section (geometry) and properties (stress-strain, strength, etc.), the relation between moment and curvature may be obtained from a FB-Pier analysis. The FB-pier fiber model considers the nonlinear stress-strain behavior of both concrete and steel and is capable of generating both a moment-curvature as well as biaxial bending diagram for any cross-section. To perform the analysis, a column with a given cross-section is specified. Next, springs of very high stiffness (both in translation and rotation) are applied to the bottom of the shaft with various moments applied to the top of the shaft (Figure 4.5). As the moment is constant along the length of the column,
curvature, $\phi$, is also constant and can be calculated as $\phi = \theta/L$, where $L$ is the length of the column (Figure 4.5). By changing the applied moment, $M$, a series of corresponding $\phi$ values or $M$ vs. $\phi$ may be obtained. Figures 4.6 and 4.7 are the $M$ vs. $\phi$ plots for the 6’ and 9’ prototype shafts used in this research.

Figure 4.5 FB-Pier Column Analyses for Moment-Curvature

To validate the analytical Moment-Curvature Relationship, $M$-$\phi$ (Figures 4.6 and 4.7), model drilled shafts of same cross-section and properties were constructed with several pairs of strain gages (Figure 4.8). Only the bottom portions of the models were cast into the rock, not containing instrumentation with strain gages. A point load was applied to the top of each shaft. From Equations 4.4 and 4.5, the Moment-Curvature, $M$-$\phi$, was computed and plotted in Figures 4.6 and 4.7.

To ensure similar Moment-Curvature Relationship, $M$-$\phi$, for each lateral load test, one pair of the 6 pair of strain gages (Figure 4.2), was located just above the rock surface. As there was no rock at the top gage level, there was no contribution from skin friction to the moment and the moment was simply equal to the applied load times the distance from
top gauges to load cell. Again, the Moment-Curvature Relationship shown in Figures 4.6 and 4.7, were determined from Equations 4.4 & 4.5.

Figure 4.6 Moment Curvature Relationship for 6ft Diameter Shaft

Figure 4.7 Moment-Curvature Relationship for 9ft Diameter Shaft
Evident from Figures 4.6 and 4.7, the Moment-Curvature, $M-\phi$ relationships obtained from the experimental methods were close. The latter suggested the shaft reinforcement, instrumentation, and construction were very repeatable. It was found that the FB-Pier column analysis was very sensitive to the properties of concrete and steel, as well as the location of the steel. The properties of the steel and concrete were varied in the column analysis until a good agreement was found between the $M-\phi$ plots from all three methods. The same properties will be used subsequently in FB-Pier to verify the calculated P-Y curves for the rock.

### 4.2.2.4 Fitting trend-lines to moment–depth graphs

Having established moment curvature-relationships, moments at each strain gage location was determined from bending strains (Equation 4.4), followed by curvature (Equation 4.5) and then moment (Figures 4.6 and 4.7). For each load case, the moment along the shaft was plotted with depth ($z$) as shown in Figure 4.9. Evident from the plot,
the moment at the top of shaft (point where load is applied) is zero and increases linearly with depth until the rock surface is reached. The moment below the rock surface increases to a maximum and then diminishes to zero at the tip of the shaft. The location of maximum moment corresponds to zero shear (i.e., shear reversal) and is the point of rotation of the shaft within the rock.

Figure 4.9 Moments vs. Depth for a Given Applied Lateral Load

A trend-line was fitted to each M-z graph. The trend line was required because the back computed P-Y curve is obtained from a second order derivative of the M-z graph. Several different trend-lines were considered for the moment depth graph but it was decided to use a third order polynomial. Higher order polynomials gave better fits to the experimental data; however their differentiation resulted in very odd looking shear and
rock P values. The third order moment distribution gives a quadratic shear distribution and a linear rock resistance (P) with depth. Figures 4.10 and 4.11 respectively show the distribution of shear force and lateral rock resistance (P) with depth, for the moment distribution presented in Figure 4.9. In addition to satisfying the calculated moments along the length of the shaft, the moment distribution had to satisfy the condition of applied shear at the top and zero shear at the tip. The use of styro-foam at the tip guaranteed a zero shear force at the shaft tip.

Figure 4.10 Shear Force Distribution with Depth
4.2.3 Rock’s Lateral Resistance $P$ (F/L) from Bending Moments and Skin Friction

The difference in the moment at two different elevations is caused by rock’s lateral (P force/length) and axial force (T force/length) resistance at the rock-shaft interface (see Figure 1.2, Chapter 1). The contribution to moment in the case of the latter is a function of shaft diameter, and the rock’s T-Z curve as well as the rotation of the shaft. Current Software (FB-Pier, LPILE, etc.) neglect the contribution of T (Figure 4.12), in the Moment Equilibrium for the resulting Shear, $V$ at a cross-section,

$$V = \frac{dM}{dz} \quad \text{Eq. 4.6}$$

Consequently, from lateral force equilibrium (Figure 4.12), the soil lateral $P$ (force/length) is found as

$$P = \frac{dV}{dz} = \frac{d^2 M}{dz^2} \quad \text{Eq. 4.7}$$
Figure 4.12  Force and Moment Equilibrium on a Shaft Element of Length $dz$

If the side shear, $T$ (Figure 4.12), is taken into account, then moment equilibrium results in

$$\frac{dM}{dz} = V + \frac{T}{\Delta z} \quad \text{Eq. 4.8}$$

or

$$\frac{dM}{dz} = V + M_s \quad \text{Eq. 4.9}$$

Where, $M =$ moment on cross-section and $M_s =$ moment per unit shaft length from the side shear fore, $T$.

Horizontal force equilibrium on the element gives

$$P = \frac{dV}{dz} \quad \text{Eq. 4.10}$$

From Equation 4.9 and Equation 4.10, then the rock lateral resistance, $P$, is obtained as:

$$P = \frac{d^2M}{dz^2} - \frac{d(M_s)}{dz} \quad \text{Eq. 4.11}$$

Evident from Equation 4.11 vs. Equation 4.7, the side shear on the shaft will reduce the rock’s lateral resistance, $P$, calculation. The latter suggests that for large diameter
drilled shaft field tests in stiff rock, the back computed P-Y curve from Equation 4.7 may be un-conservative.

The moment/unit length, Ms, of the side shear is obtained from the T-Z curve for the rock. The value of T requires the displacement, z, at a point on the shaft. From the strain gage data, the angle of rotation (θ) will be determined and then the axial displacement (z) will be computed using the shaft’s diameter. Also for the development of the P-Y curve, the lateral displacement, Y, is needed. The latter will be found from the shaft rotation, θ, and the top shaft displacements. The computation of Ms used to find P is undertaken in the next section.

4.2.4 Moment Due to Side Shear, Ms

Lateral loading causes a rotation of the shaft at any given cross section. The shaft rotation is resisted through skin friction, T, and lateral rock resistance, P, acting on the sides of the shaft. In the case of the unit skin friction, a Moment/length resistance, Ms, may be computed at any cross-section. The value of Ms is a function of the unit skin friction at the periphery of the shaft, which varies around the shaft’s circumference.

To estimate the moment due to side shear (Ms), the shaft cross section was divided into slices as shown in Figure 4.13. For this idealization, the shaft is loaded in direction AC. All the points to the left of line BD move up while those on the right of BD are assumed to move down. Next, the cross-section of the shaft was cut into 10 slices, (Figure 4.13). R_i is the distance from the centre of shaft to the center of slice i. For example R_1, is the distance from the center of the shaft to the middle of slice 1(shaded).
Next, the sum of arc lengths BP and DQ is referred to as $C_1$ where subscript 1 refers to slice 1. Note, both arcs are summed together, i.e., BP or DQ, since the shear stress, $\tau$, is assumed constant on both sides of the slice. The value of shear stress, $\tau$, is a function of vertical displacement, $z_i$, which is a function of the rotation, $\theta$, and the distance from the center of cross-section to the center of the slice, $R_i$.

If $z_1$ is the average axial displacement of slice 1 and $\tau_1$ (obtained from T-Zcurve knowing $z_1$) then the side shear force/unit length, $T_1$, acting on slice 1 is given by

$$T_1 = \tau_1C_1$$  \hspace{1cm} \text{Eq. 4.12}$$

The moment per unit shaft length about O, $M_{s1}$, is found by multiplying $T_1$ by the distance to the cross-section centroid, $R_1$, as

$$M_{s1} = T_1R_1 = \tau_1C_1R_1$$  \hspace{1cm} \text{Eq. 4.13}$$

The total moment per unit length may be found by summing the moments acting on all the slices:
\[
M_s = \sum_{i=1}^{n} \tau_i C_i R_i 
\]
Eq. 4.14

where \(n\) = number of slices.

In the above estimation of \(M_s\), it is assumed that the neutral axis (i.e., center of rotation) of the shaft remains at the center of the cross section. Although this assumption is not the case for large strains, the effect of changing the position of neutral axis was found to be small on \(P\) estimation. For instance, an analysis of a 9 ft diameter shaft with an applied lateral load of 5000 kips at 7 ft above rock surface with FB-Pier showed a maximum movement of the neutral axis of 10 inches. Subsequent calculation of \(M_s\) with the cross-section’s center of rotation at the centroid or at the neutral axis made only a 1% difference in the calculation of \(P\) (force/length).

The \(M_s\) was developed (Equation 4.14) at every strain gage location along with a trend line with depth. Using Equation 4.11 with trend lines for both \(M\) and \(M_s\), the rock’s lateral resistance, \(P\) (F/L), was determined as function of applied lateral load.

### 4.2.5 Lateral Deflections, \(Y\) for a Given Rock Resistance, \(P\)

Having obtained the rock resistance, \(P\), the corresponding lateral displacement, \(Y\), is required to obtain a \(P-Y\) curve. The distribution of \(Y\) with depth (i.e., the deflected shape of the shaft) was determined as follows.

Figure 4.14 shows the deflected shape of a shaft for a given applied lateral load at the top of the shaft. With depth on the sides of the shaft are pairs of strain gages labeled B to G, Figure 4.14. At the top of the shaft, A (Figure 4.14) the lateral displacement from LVDT measurement is known. Also at some depth below the rock surface there is point on the shaft which undergoes no lateral displacement, \(y = 0\), (point O in Figure 4.14). That is above this point, the shaft is moving one way, and below this point, the shaft is
moving the other way. Referred to as the point of zero deflection, \( y = 0 \), it corresponds to zero lateral resistance, \( P \), on the P-depth distribution curve (Figure 4.11), and is known.

![Figure 4.14 Computing Deflected Shape of the Shaft with Depth](image)

Next, from the strains at individual cross-sections B, C, …G, the rotations are found, and the average rotation of segments, BC, CD, … FG (\( \Delta\theta_1 \) through \( \Delta\theta_5 \), Figure 414) was determined,

\[
\Delta\theta = \frac{\varepsilon_{bt} + \varepsilon_{bb}}{2r} \times L
\]

**Eq. 4.15**

where, \( \varepsilon_{bt} \) and \( \varepsilon_{bb} \) are the bending strains at the top and the bottom of each segment respectively, \( r \) is the shaft radius and \( L \) is the segment length.
\[ \frac{E_{bl} + E_{bb}}{2r} \]

The quantity \( \frac{E_{bl} + E_{bb}}{2r} \) represents the average curvature over a segment and when multiplied by the segment length gives the average change in angle over the segment.

For the lateral displacement calculation, at least one absolute value of \( \theta \) (i.e., relative to fixed reference) is needed in order to calculate the absolute value of \( \theta \) at all 6 strain gage positions. Two LVDTs were used to measure lateral displacements at two points above the rock surface. These measured displacements could be used to estimate the angle \( \theta_1 \) at the rock surface. However, since the difference between these two LVDT readings was small and very sensitive to noise, the lateral deflections calculated from the LVDTs did not always satisfy the condition of zero deflection at \( P=0 \). It was, therefore decided to use only one measured displacement and an iterative method to obtain the lateral deflections. The method used is described:

1. An estimate of shaft rotation at the top strain gage position (\( \theta_1 \)), i.e., rock surface was found by drawing a straight line from point A to the point of zero deflection (O), as shown in Figure 4.14.

2. Using the latter estimate of \( \theta_1 \), the segment rotations, \( \Delta \theta \): Eq. 4.15, the rotations (\( \theta_i \)) at each point on the shaft, i.e., C to G was calculated (i.e., \( \theta_2 = \theta_1 - \Delta \theta_1 \), \( \theta_3 = \theta_2 - \Delta \theta_2 \) and etc.)

3. The computed \( \theta_i \) (step 2) at the strain gage positions represent the slopes of the deflected shape at their respective positions (\( dy/dz_i = \theta_i \)). Hence a smooth line was drawn from point A (\( y = y_0 \)) with slopes at the strain gage positions (B to G) equal to \( \theta_i \).

4. Steps 1 to 3 were repeated until the line passed through or was very close to the point O (point of zero deflection and \( P = 0 \)).
Shown in Figure 4.15 is a typical displacement vs. depth (prototype dimensions) for a shaft using the method described above. The moment, shear and rock lateral pressure are for the same load case as was presented in Figures 4.9 to 4.11.

Figure 4.15 Typical Lateral Deflection (y) vs. Depth Plot
CHAPTER 5
RESULTS

5.1 Axial Skin Friction

Axial load tests were conducted on drilled shafts embedded in the rock at 3 different rock strengths: 10tsf, 20tsf, and 40tsf. For the 10tsf strength rock, the axial load tests were repeated three times (Figure 5.1), and for the two higher strengths (20tsf and 40tsf), the tests were repeated twice (Figures 5.2 and 5.3). All of the tests were performed on 6’ diameter shafts embedded 18’ (L/D = 3) into the rock. All of the plots, (Figures 5.1 - 5.3) show the load – displacement data which mobilize significant axial resistance with small displacements (i.e., 80% capacities at 0.5% of diameter). Axial load test in lower strength 5tsf rock, proved unattainable, because the rock mass fractured from the shaft to the boundaries of the bucket.

Figure 5.1 Axial Load vs. Displacement in 10 tsf Strength Rock
The load applied at the top of each shaft was subsequently converted into shear stress (skin friction, $f_s$) on the shaft/rock interface by dividing by the shaft area (as described in Chapter 4). Since styro-foam was placed at the shaft tip, the entire load was...
transferred to the rock through skin friction. Plots of $f_s$ vs. axial displacement (T-Z curves) for each strength rock are shown in Figure 5.4. The data points were obtained by averaging the multiple centrifuge tests for each strength. Also shown in the plot are trend-lines for each rock strength.

![Figure 5.4 T-Z Curves for 10, 20 and 40 tsf Rocks](image)

From the T-Z curves, the ultimate unit skin frictions were established from the horizontal tangents. Ultimate unit skin friction values of 53psi, 92psi and 160psi, were found for rock strengths of 10tsf, 20tsf, and 40tsf, respectively. Of interest is the comparison of the measured unit skin frictions and those predicted based on FDOT design equation (McVay,1992)

$$f_s = \frac{1}{2} \sqrt{q_u \cdot q_t}$$  \hspace{1cm} \text{Eq. 5.1}

where $q_u$ and $q_t$ are the unconfined compression and split tensile strengths.
Table 5.1 shows the comparison of the ultimate unit skin friction predicted from Equation 5.1 with the measured centrifuge values. Evident, from the table, the measured values are higher than predicted values from Equation 5.1. Suggested reasons for their difference are 1) use of synthetic rock vs. Florida Limestone, and 2) method of construction. In the case of the synthetic rock, bentonite slurry was used to separate the shaft concrete from the synthetic rock, which contained cement, which chemically bonded to the shaft. It is postulated that the bentonite coating could have been scrapped off in some zones along the shaft during construction. Further, as identified earlier (Chapter 3), the hole in the synthetic rock for the model construction was always larger (0.01”) at the top of the shaft vs. the bottom. The latter through scaling (N=67) is approximately 1” in prototype dimensions. The latter leads to increased horizontal stress and unit side friction on the shaft-rock interface. For computing the P-Y curves (section 5.2), the measured unit skin curves, T-Z were used.

Table 5.1 Comparison of Measured and Predicted Ultimate Skin Friction for Synthetic Limestone

<table>
<thead>
<tr>
<th>Rock qu (tsf)</th>
<th>Rock qt (tsf)</th>
<th>Ultimate unit skin friction (tsf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5(qu*qt)^0.5</td>
<td>Measured</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>1.6</td>
<td>2.0</td>
</tr>
<tr>
<td>20</td>
<td>3.4</td>
<td>4.1</td>
</tr>
<tr>
<td>40</td>
<td>5.6</td>
<td>7.5</td>
</tr>
</tbody>
</table>

To compute the P-Y curves, T-Z curves independent of diameter are required.

Shown in Figure 5.5 are the normalized T-Z curves (Figure 5.4): \( f_s \) values were normalized with respect to \( f_{s_{\text{max}}} \) (ultimate unit skin friction) and vertical displacement, \( z \), was normalized with respect to \( D \) (diameter). Evident from the figure, the three
normalized curves are quite similar and can be represented by a single curve (shown in
bold line), with the following equations:

\[
\frac{f_s}{f_{s_{\text{max}}}} = 0.96R^{0.33} \quad 0 \leq R \leq 0.5 \quad \text{Eq. 5.2}
\]

\[
\frac{f_s}{f_{s_{\text{max}}}} = 0.86R^{0.16} \quad 0.5 \leq R \leq 3 \quad \text{Eq. 5.3}
\]

\[
\frac{f_s}{f_{s_{\text{max}}}} = 1 \quad 3 \leq R \quad \text{Eq. 5.4}
\]

where, \( R = \frac{z}{D}\times 100 \).

Figure 5.5 Normalized T-Z Curves for Synthetic Rock

Kim (2001) analyzed data from 33 axial load tests (Osterberg) from various bridge
sites throughout Florida and recommended the normalized T-Z curve for the natural
Florida Limestone given in Figure 5.6. A comparison of Kim’s normalized T-Z curve
with the synthetic rock curve (Figure 5.5) is also shown in Figure 5.6. Evident from the
figure there is a very good agreement between the normalized T-Z behavior of the natural limestone and the synthetic rock. Figure 5.6 will be used in the P-Y curve estimation.

Figure 5.6 Comparison of Normalized T-Z Curves, Synthetic Rock vs. Natural Florida Limestone

### 5.2 Lateral P-Y Resistance

The methods and assumptions used in calculating P-Y curves were discussed and presented in Chapter 4. As identified, two types of P-Y curves were back computed: 1) P-Y curves considering unit side shear, Equation 4.11, and 2) P-Y curves neglecting all side shear on the shafts, (Equation 4.7). All back computed curves were obtained from the 12 lateral load tests performed in the centrifuge with diameters of 6 and 9ft, embedment (L/D) of 2, 3 and 4 and rock strengths of 10 and 20 tsf. Note that each lateral load test gave multiple P-Y curves, which were averaged to obtain a representative curve (Figure 5.7).
5.2.1 Back Computed P-Y Curves Neglecting Side Friction

Figure 5.7(a) shows the typical back computed P-Y curves for a 6ft diameter shaft embedded in 10 tsf rock. The depth of embedment for this shaft was 18 ft (i.e., L/D ratio of 3). The P values have been divided by $q_u D$ (rock compressive strength multiplied by shaft diameter) and the y values by $D$ to make the curves dimensionless. The points that are plotted were calculated for the whole shaft from the rock surface to the shaft tip for each case. As evident in Figure 5.7(a), the points have some scatter with the deeper curves (i.e., not as large P values) being softer than the upper curves (high P values). It is believed the scatter is due to just the third polynomial fit to the moment distribution along the shaft (i.e., linear for P: i.e., secant lines) as well the accuracy of the instrumentation. Consequently, representative trend-line was drawn through the middle of all the points (Figure 5.7(b)).
As part of the P-Y curve development/validation, the curves were inputted to FB-Pier and used to predict the moments, displacements, etc. for each experiment. Note, in addition to the standard P-Y curves such as soft clay model, stiff clay model, FB-Pier has provision for input of custom P-Y curves. It was expected that the back computed P-Y should be able to simulate the results of a centrifuge experiments. The shaft steel and concrete properties used in the modeling were the same as those used to obtain the moment curvature relations for calculating moments from strain data, (Chapter 4). The resulting deflections of the shaft were compared with the actual deflections obtained from test for different load cases. Figure 5.8 shows the comparison of deflections for the shaft described above, at four different lateral load cases. Evident from the figure, the agreement in displacement was poor at small loads, but gave quite similar response at high loads. In particular, the back computed P-Y curves were too soft initially. The latter could be a result of the accuracy of the approach to estimate the lateral displacement, Y, of the shaft (section 4.2.5) at small lateral displacements.

Consequently, it was decided to adjust the P-Y curves in Figure 5.7(b) to better fit the measured centrifuge results with FB-Pier. This was done by altering the initial shape of the P-Y curves until FB-Pier gave a good match of deflections. The resulting P-Y curve is given in Figure 5.9 along with the original curve for comparison. The predicted (using adjusted P-Y) and measured deflections of the shaft under various lateral loading are given in Figure 5.10. The same four load cases of Figure 5.8 were used. Evident from the plots, the agreements between measured and predicted deflections under various loads are excellent. Note that only the initial portion of the P-Y curve was adjusted. Figure 5.11 presents the back computed P-Y curves for the 6’ and 9’ diameter shafts embedded
in the 10 tsf and 20 tsf strength (qu) rock. Evident from the figure, the curves are a function of rock strength and shaft diameter.

Figure 5.8 Comparison of Deflections (Test vs. FB-Pier Modeling) for 4 Load Cases
5.2.2 Comparison of Measured and Published P-Y Curves Neglecting Side Friction

Reese and Nyman performed lateral load tests in 1978 on instrumented drilled shafts cast in vuggy limestone in the Florida Keys. The shafts were 4’ in diameter, 44’ long, with 12’ of soil overburden. The rock had an unconfined compressive strength of 15tsf. Unfortunately, the tests were conducted to a lateral load of only 75 tons, with a lateral displacement of 0.0213” at the top of the rock. In addition, the shafts were modeled as linear medium (i.e., constant M-θ). Figure 5.12 shows the Reese and Nyman curve for a drilled shaft of diameter B in rock of undrained shear strength of Su, which terminates well before the measured curve goes nonlinear.
Figure 5.10 Measured and Predicted Lateral Deflections of Shaft under Lateral Load (Modified P-Y)
Figure 5.11 Measured and Predicted P-Y Curves without Side Shear

Figure 5.12 P-Y Curve for Limestone, Reese and Nyman, 1978
Due to a lack of published P-Y representation for Florida Limestone, a number of consultants have employed either the soft or stiff clay models with the rocks’ strength characteristics (i.e., $q_u$ and $q_t$). A brief discussion of each follows along with a comparison with the measured response. A complete description of the curves can be found in FHWA’s COM624 manual.

The soft clay model is one of the standard P-Y curves used in FB-Pier to model lateral behavior piles/drilled shafts. The soft clay model, developed by Matlock (1970) uses the following equation to calculate the points on the P-Y curve:

$$\frac{P}{P_u} = 0.5 \left( \frac{y}{y_{50}} \right)^{0.33} \text{ for } 0 \leq y \leq 8y_{50}$$  \hspace{1cm} \text{Eq. 5.5}

$$\frac{P}{P_u} = 1 \text{ for } y > 8y_{50}$$  \hspace{1cm} \text{Eq. 5.6}

where, $P_u$ = ultimate soil resistance per unit pile/shaft length and is equal to the smaller of the values given by Equations 5.7 and 5.8.

$$P_u = \left( 3 + \frac{\gamma'}{C} H + \frac{J}{D} H \right) CD$$  \hspace{1cm} \text{Eq. 5.7}

$$P_u = 9CD$$  \hspace{1cm} \text{Eq. 5.8}

and $\gamma'$ = average effective unit weight of soil from ground surface to P-Y curve,

$H$ = depth from ground surface to P-Y curve,

$C$ = shear strength of rock at depth $H$,

$D$ = pile/shaft width (diameter if circular)

$J = 0.25$ for stiff clay/rock (found by Matlock, 1970 experimentally).

$$y_{50} = 2.5\varepsilon_{50} D$$  \hspace{1cm} \text{Eq. 5.9}

where, $\varepsilon_{50}$ = strain at half the maximum principal stress difference.
The method of computing P-Y curve for stiff clay was developed by Welch and Reese (1972). The lateral resistance, P, as a function of lateral displacement is given by

\[ \frac{P}{P_u} = 0.5 \left( \frac{y}{y_{50}} \right)^{0.25} \quad \text{for} \quad 0 \leq y \leq 16y_{50} \]  
Eq. 5.10

\[ \frac{P}{P_u} = 1 \quad \text{for} \quad y > 16y_{50} \]  
Eq. 5.11

where \( P_u \) and \( y_{50} \) are given by Equations 5.7, 5.8 and 5.9 respectively.

Figure 5.11 also shows both the stiff and soft clay models’ predicted P-Y curves using the measured rock strength. Evident, both models under predict the Florida Limestone’s lateral resistance.

5.2.3 Back Computed P-Y Curves Corrected for Side Friction

The process of obtaining P-Y curves from moment data corrected for shaft side friction is described in Chapter 4. Shown in Figure 5.13 is a typical back computed P-Y curve (original) for a 6’ diameter, 18’ embedded shaft in 10tsf strength rock. The back computed P-Y curve was then used in FB-Pier to predict the experimental results. However, since the current version of FB-Pier does not employ rotational springs representing the shaft’s side shear, a moment equal to the side shear (see Equation 4.13, Chapter 4) over the shaft element (section of shaft between strain gages) was inputted. Note, the moment applied along the length of the shaft at the end nodes of each element was varied for each load case. Figure 5.14 shows the lateral displacements along the shaft for various applied lateral loads. Evident, from the figure, the predicted displacements are too large for small lateral loads and agree well at the higher loads. Consequently the back computed P-Y curve (Figure 5.13), was modified until better agreement for lower lateral loads (Figure 5.15) were attained. Comparison of displacements for all 12 lateral
tests using original and modified P-Y curves both with and without side shear is presented in Appendix B.

A comparison of the adjusted P-Y curves which either ignore (Figure 5.9), or consider unit side shear (Figure 5.13), reveal a reduction in P-Y resistance with side shear for the same strength rock (10 tsf). The latter is expected, since a portion of the shaft’s lateral resistance (Figure 5.13), is being carried by unit side shear.

Figure 5.13 Modified and Original P-Y Curves, With Shaft Side Shear

Presented in Figure 5.16 are back-adjusted P-Y curves for all twelve-centrifuge tests with side shear considerations; i.e., two shaft diameters (6’ and 9’), three embedment lengths (L/D =2, 3, & 4) and two rock strengths (10 tsf and 20 tsf). Also shown in the figure are the predicted P-Y curves for soft and stiff clay models. Evident from the figure, even though the lateral resistance is normalized with rock strength and
diameter, there is quite a bit of variability in the P-Y curves. Consequently, an attempt to normalize the P-Y curves was undertaken.

Figure 5.14 Measured and Predicted (Original P-Y) Lateral Deflections of 6’ Diameter Shaft in 10 tsf Rock
Figure 5.15 Measured and Predicted (modified P-Y) Lateral Deflections of 6’ Diameter Shaft in 10tsf Rock
5.2.4 Normalized P-Y Curves

It is common practice to normalize P-Y curves with $q_u D$ and $D$ as was done for both the inclusion and exclusion of side friction (Figures 5.11 and 5.16). However, the computed P-Y curves for rock still display significant variability as a function of rock strengths and shaft diameters. Hence, better normalizing functions were needed for both curves (Figures 5.11 and 5.16). After a number of trials, it was found that, the uncorrected (Figure 5.11), and the corrected curves (Figure 5.16), could be represented by a single trend-line if the $P$ values are normalized with $q_u^{0.25} D^{0.9}$ and $q_u^{0.15} D^{0.75}$ respectively. Figures 5.17 and 5.18 present the normalized P-Y curves for Florida Limestone with the inclusion or exclusion of shaft side friction. Note the curves are valid...
for all the experimental results (i.e., 6’ and 9’ diameter shafts, different rock strengths, etc.).

Figure 5.17 Normalized P-Y curves Without Side Shear Correction

Figure 5.18 Normalized P-Y Curves Corrected for Side Shear
Note however, the P-Y curves are unit dependent. That is, the rock unconfined compressive strength \((q_u)\), the shaft diameter and rock’s lateral resistance, \(P\) must be in ksf, feet and kips/ft respectively.

5.2.5 Influence of Rock Strength, and Shaft Diameter on P-Y Curves

For this research, the P-Y curves were found from shafts with 6ft and 9ft diameters and rock strengths of 20 and 40 ksf. However, with trend-lines given in Figures 5.17 and 5.18, P-Y curves may be reconstructed for any shaft diameter and rock strength representing field conditions. For example, the graphs in Figure 5.19 show P-Y curves reconstructed for 3ft, 6ft, 9ft and 12ft diameter drilled shafts embedded in 80 ksf rock. The latter diameters are typical in many Florida drilled shaft design/construction projects. Figure 5.20 presents the reconstructed P-Y curves for a 6 ft diameter shaft embedded in 10, 20, 40 and 80 ksf rock. In both figures, the percentage difference (as percentage of the uncorrected value) between corrected (considers side shear) and uncorrected (neglects side shear) P values are also shown along with the curves. In Figure 5.20, the P-Y curves obtained from the centrifuge tests are plotted for 20 and 40 ksf rock for comparison.

It is evident from Figures 5.19 and 5.20 that the reduction in \(P\) due to side shear is more dependent on rock strength than on shaft diameter. For example, increasing the diameter from 3 to 12 ft results in 5% reduction in the P-Y curve, whereas, increasing the rock strength from 20 to 80 ksf reduced the P-Y curve by 12%.
Figure 5.19 Reconstructed P-Y Curves for 80 ksf Rock for Various Diameter Shafts
Figure 5.20 Reconstructed P-Y Curves of a 6ft Diameter Shaft in Various Strength Rock

5.2.6 Recommended Moment-Rotation Springs for FB-Pier

In the new FB-pier code, it is proposed to model the moment due to axial side shear by rotational springs (Ms-θ) at the pile/shaft nodes in addition to the existing P-Y and T-Z springs. The process of obtaining P-Y curves involved the calculation of the angle of
rotation ($\theta$) and the moment ($M_s$) per unit shaft length at strain-gage elevations. A plot of $M_s$ against $\theta$ is a $M_s$-$\theta$ curve. $M_s$-$\theta$ curves for both the 6’ and 9’ diameter shafts embedded in the 10 tsf and 20 tsf rock are shown in Figures 5.21 and 5.22.

![Figure 5.21 Moment from Shaft Side Shear vs. Rotation for 10 tsf Rock](image)

In Figure 5.23 all $M_s$-$\theta$ curves have been normalized with the maximum available moment ($M_{s\text{max}}$), which is related to the ultimate skin friction ($f_{su}$) of rock and the shaft diameter ($D$). Curves representing all the tests (2 diameters and 2 rock strengths) are plotted and a single curve (trend-line) is drawn, which can be expressed by the following equations:

$$\frac{M_s}{M_{s\text{max}}} = 3.49\theta^{0.33} \quad \text{for } 0 \leq \theta \leq 0.01 \text{ rad} \quad \text{Eq. 5.12}$$

$$\frac{M_s}{M_{s\text{max}}} = 1.61\theta^{0.33} \quad \text{for } 0.01 \leq \theta \leq 0.06 \text{ rad} \quad \text{Eq. 5.13}$$

$$M_{s\text{max}} = \frac{f_{su} \pi D^2}{3.35} \quad \text{Eq. 5.14}$$
Figure 5.22 Moment from Shaft Side Shear vs. Rotation for 20 tsf Rock

Figure 5.23 Normalized Moment ($M_s$)–Rotation ($\theta$) Curve
Note that Figures 5.21 and 5.22 are for no applied axial forces on the shaft. They may be used in combination with Equation 4.11 to back compute P-Y curves from field lateral load tests to account for shaft side shear. The latter is recommended, since the side shear will reduce back computed P-Y curves by 25% to 30%, making the conventional curves unconservative. It is also recommended that full-scale field tests be employed to validate the P-Y curves developed in Figures 5.17 and 5.18.
CHAPTER 6
SUMMARY AND RECOMMENDATIONS

6.1 Summary

For axial loading, T-Z curves characterize the axial skin friction on a drilled shaft as a function of its vertical load and displacement. For many pile/shaft programs, i.e., FB-Pier, Lpile, etc., the side Friction, T (Figure 6.1) is assumed to act through the center of the shaft, i.e., point A. Therefore, in the case of lateral loading, the shear, V, and moment, M, in a segment of shaft (Figure 6.1) is assumed to be resisted by the soil/rock lateral resistance, P (Force/unit length) with no contribution of T (Figure 6.1) occurring.

Figure 6.1 Forces & Moments Acting on Segment of Drilled Shaft

In the case of large shafts, or strong rock, the side shear, T (Figure 6.1) may develop a significant couple (Moment, M,) depending on rotation, θ, of the cross-section and if T is assumed to act at the rock/shaft interface, Figure 6.1. The latter may have a
significant effect on both the Moment, M, and the back computed P-Y curve for a laterally loaded shaft. The rotational spring, M_s - θ, as well as P-Y curves for Florida Limestone for FB-Pier implementation was the focus of this research.

For this study, 7 axial and 12 lateral tests were conducted in a centrifuge on drilled shafts embedded in variable strength rock to develop typical P-Y, T-Z and M_s-θ curves for Florida limestone. Synthetic limestone was used to ensure repeatability of tests as it was not viable to obtain uniform samples of natural limestone. Synthetic limestone, made by mixing ground up natural limestone, cement and water was believed to be representative of natural limestone. The two have been shown to have similarity in their strength and stiffness characteristics.

Results of the axial tests showed the skin friction of synthetic rock was higher than Florida limestone of similar strength. This was attributed to the construction technique employed in the model construction (e.g., the hole for casting model shaft was narrower at the bottom than at the top) and the chemical properties of the synthetic rock (e.g., bonding of shaft concrete with rock). However, T-Z curves normalized with the ultimate unit friction show a remarkable similarity to the trend observed from the results of 33 Osterberg tests at various sites in Florida.

Results from 12 lateral tests conducted on 2 rock strengths, 2 shaft diameters and 3 L/D ratios were used to back calculate P-Y curves for Florida Limestone. P-Y curves were found if the shaft side friction, T (Figure 6.1), was considered or not. Subsequently, all the curves were normalized and are shown in Figures 6.2 and 6.3.
Figure 6.2 Normalized P-Y curves Without Side Shear Correction

The normalized back computed P-Y curve with (Figure 6.3) and without (Figure 6.2) side friction, show that the current use of either the soft or stiff clay models to characterize Florida Limestone, are conservative. However, the current practice of neglecting side friction in back computing P-Y curves is unconservative as shown in Figure 6.4. The plot shows a typical 6’ diameter drilled shaft embedded in 40 tsf rock has a P-Y curve which is 25% too high if side friction is neglected. That is a portion of the P-Y curve or lateral resistance which is assumed to be from rock shear strength, is actually due to axial side shear, T in Figure 6.1, along the shaft. However, if an axial load were to act on the shaft (i.e., structure live and dead loads), then the side friction, T, (Figure 6.1), would be mobilized under the axial load acting on the shaft, and would no longer be available to resist the lateral load. In addition, any lateral load test with variable axial load would generate different P-Y resistance curves (Figure 6.2).
Figure 6.3 Normalized P-Y Curves Corrected for Side Shear

Figure 6.4 P-Y Curves for 6’ Diameter Shaft, With and Without Side Friction
Also evident from Figures 6.2 and 6.3, the normalized P-Y curves are dependent on rock strength and shaft diameter but independent of shaft embedment depth, or L/D ratio. The curves were employed in FB-Pier and used successfully to predict all the experimental results. In the case of the P-Y curves, which are corrected for side friction, Figure 6.3, a moment resistance as a function of rotation, i.e., \( M_s \) vs. \( \theta \), was applied to each node in the rock. The moment due to side shear, \( M_s \), and the rotation of the shaft at strain gage locations were plotted to obtain Ms-\( \theta \) curves for both rock strengths tested as well as shaft diameters. The Ms-\( \theta \) curves closely resemble the T-Z curves and can be derived automatically from T-Z curves if the shaft rotations are known.

### 6.2 Recommendations

P-Y and Ms-\( \theta \) curves obtained in this research are based only on 2 rock strengths and 12 lateral tests. The results must be used with caution, especially outside the range of rock strengths tested. In addition, although synthetic limestone has been proved to be similar to Florida limestone in terms of its physical characteristics, its chemical properties may make it behave differently. Therefore, it is strongly recommended that field tests be performed to verify the results obtained from this study.

Having verified the results of this research by field study, it is also recommended that the new P-Y curves adjusted for side shear (Figure 6.3), as well as Ms-\( \theta \) curves be implemented in the FB-Pier software to model the actual axial and lateral transfer of the pile/drilled shaft load to the rock. For conservative design, the adjusted side shear P-Y curves (Figure 6.3) should be used with no Ms-\( \theta \) springs. The latter would assume all shaft side shear (Figure 6.1), was being fully mobilized under axial conditions with none available for lateral loading.
This section presents the raw data from the 12 lateral load tests, which were used to back compute P-Y curves. For each test a graph of load vs. displacement and 6 graphs of load vs. strain are plotted. Each load vs. strain plot consists of strains on the opposite sides of the shaft at a given depth.

The load, displacement and strains shown in the graphs are those for the model shaft. Since the tests were conducted at a centrifugal acceleration of 67g, loads, displacements and strains have to be multiplied by appropriate scaling factors (Table 2.1, Chapter 2) to obtain the values for a prototype shaft as follows:

- Scaling factor $N = 67$
- Displacement (linear dimension) is scaled $1: N$.
  Therefore, prototype displacement = $67 \times$ model displacement.
- Load (force) is scaled $1: N^2$.
  Therefore, prototype load = $4489 \times$ model load.
- Strain is scaled $1:1$.
  Therefore, prototype strain = model strain.
Lateral Test# L1

Shaft Information and Load- Displacement Data

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Diameters

| Shaft | 1.080 6.03 |
| Rebar cage | 0.750 4.19 |

Load- Strain Data
Lateral Test# L2

Shaft Information and Load-Displacement Data

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Diameters

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| Rebar cage | 0.750 | 4.19 |

Load- Strain Data
Lateral Test# L3

Shaft Information and Load-Displacement Data

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Diameters

| Shaft | 1.060 | 5.92 |
| Rebar cage | 0.750 | 4.19 |

Load- Strain Data
Lateral Test# L4

Shaft Information and Load-Displacement Data

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<td>LVDT2</td>
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<td>6.33</td>
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<tr>
<td>strain g1</td>
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<td>6.33</td>
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<tr>
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<tr>
<td>strain g3</td>
<td>2.550</td>
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<td>3.259</td>
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<td>strain g5</td>
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<td>tip</td>
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<td>28.09</td>
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Diameters

<p>| | | |</p>
<table>
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<tr>
<th></th>
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<tbody>
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<td>5.92</td>
</tr>
<tr>
<td>Rebar cage</td>
<td>0.750</td>
<td>4.19</td>
</tr>
</tbody>
</table>

Load-Strain Data

Strains at g1

Strains at g2
Lateral Test# L5

Shaft Information and Load-Displacement Data

<table>
<thead>
<tr>
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<tbody>
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<td>LVDT</td>
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<td>Load</td>
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</tr>
<tr>
<td>strain g1</td>
<td>1.183</td>
</tr>
<tr>
<td>strain g2</td>
<td>1.734</td>
</tr>
<tr>
<td>strain g3</td>
<td>2.285</td>
</tr>
<tr>
<td>strain g4</td>
<td>2.837</td>
</tr>
<tr>
<td>strain g5</td>
<td>3.388</td>
</tr>
<tr>
<td>strain g6</td>
<td>3.939</td>
</tr>
<tr>
<td>tip</td>
<td>4.215</td>
</tr>
</tbody>
</table>

Diameters

| Shaft | 1.060 | 5.92 |
| Rebar cage | 0.750 | 4.19 |

Load-Strain Data
Lateral Test# L6

Shaft Information and Load-Displacement Data

<table>
<thead>
<tr>
<th>qu (ksf)</th>
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<tbody>
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<td>Elevation</td>
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<td>Load</td>
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<td>strain g1</td>
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<td>strain g2</td>
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<td>strain g3</td>
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<td>strain g6</td>
<td>3.939</td>
</tr>
<tr>
<td>tip</td>
<td>4.215</td>
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</table>

Diameters

| Shaft     | 1.060  | 5.92  |
| Rebar cage| 0.750  | 4.19  |

Load-Strain Data

![Graphs showing load-strain data for g1 and g2 at various strains and loads.](image-url)
Lateral Test# L7

Shaft Information and Load-Displacement Data

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<thead>
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<td>strain g6</td>
<td>5.247</td>
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<tr>
<td>tip</td>
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</table>

Diameters

| Shaft | 1.580 | 8.82 |
| Rebar cage | 1.310 | 7.31 |

Load-Strain Data
Lateral Test# L8

Shaft Information and Load-Displacement Data

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<td>strain g2</td>
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<td>3.712</td>
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<td>5.287</td>
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<td>tip</td>
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Diameters

| Shaft     | 1.580  | 8.82 |
| Rebar cage| 1.310  | 7.31 |

Load-Strain data

Strains at g1

Strains at g2
Lateral Test# L9

Shaft Information and Load-Displacement Data

<table>
<thead>
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<td>strain g2</td>
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Diameters

| Shaft     | 1.060 | 5.92 |
| Rebar cage| 0.750 | 4.19 |

Load- Strain Data

Strains at g1

Strains at g2
Lateral Test# L10

Shaft Information and Load-Displacement Data

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Diameters

| Shaft | 1.580 | 8.82 |
| Rebar cage | 1.310 | 7.31 |

Load-Strain Data

Strains at g1

Strains at g2
Lateral Test# L11

Shaft Information and Load-Displacement Data

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<tr>
<td>strain g1</td>
<td>1.233</td>
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<td>strain g2</td>
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</tr>
<tr>
<td>tip</td>
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Diameters
- Shaft: 1.580 in, 8.82 ft
- Rebar cage: 1.310 in, 7.31 ft

Load-Strain Data

Strains at g1
- ε (microstrain) vs. Load (lb)

Strains at g2
- ε (microstrain) vs. Load (lb)
Lateral Test# L12

Shaft Information and Load-Displacement Data

<table>
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<th>Value</th>
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<tr>
<td>Load</td>
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<tr>
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<td>Diameters</td>
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<tr>
<td>Rebar cage</td>
<td>0.750</td>
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<tr>
<td>prototype(ft)</td>
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<tr>
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<td>4.19</td>
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Load-Strain Data

Strains at g1

Strains at g2

Load (lb)
APPENDIX B
P-Y CURVES AND COMPARISON OF LATERAL DISPLACEMENTS

Lateral Test# L1

P-Y Curves

Not corrected for side shear

Corrected for side shear

Original P-Y curve
Modified P-Y curve
Lateral Displacements (Not Corrected for Side Shear)

Lateral load = 1592kips

Lateral load = 1991kips

Lateral load = 2500kips

Lateral load = 3000kips

Depth (ft)

y (in)

Test

FB-Pier (modified p-y)

FB-Pier (original p-y)
Lateral Displacements (Corrected for Side Shear)

- Lateral load = 1592 kips
- Lateral load = 1991 kips
- Lateral load = 2500 kips
- Lateral load = 3000 kips
Lateral Test# L2

P-Y Curves

Not corrected for side shear

Corrected for side shear
Lateral Displacements (Not Corrected for Side Shear)

Lateral load = 1370kips

Lateral load = 2447kips

Lateral load = 3403kips
Lateral Displacements (Corrected for Side Shear)

Lateral load = 1370kips

-2 -1 0 1 2 3 4

y(in)

Depth(ft)

Test
FB-Pier(modified p-y)
FB-Pier(original p-y)

Lateral load = 2447kips

-2 -1 0 1 2 3 4

y(in)

Depth(ft)

Test
FB-Pier(modified p-y)
FB-Pier(original p-y)

Lateral load = 3403kips

-3 -2 -1 0 1 2 3 4 5

y(in)

Depth(ft)

Test
FB-Pier(modified p-y)
FB-Pier(original p-y)
Lateral Test# L3

P-Y Curves

Not corrected for side shear

Corrected for side shear

- Original P-Y curve
- Modified P-Y curve
Lateral Displacements (Not Corrected for Side Shear)

Lateral load = 1450 kips

Lateral load = 2239 kips

Lateral load = 2421 kips

Lateral load = 2636 kips
Lateral Displacements (Corrected for Side Shear)

Lateral load = 1450 kips

Lateral load = 2239 kips

Lateral load = 2421 kips

Lateral load = 2636 kips
Lateral Test# L4

P-Y Curves

Not corrected for side shear

Corrected for side shear

Original P-Y curve

Modified P-Y curve
Lateral Displacements (Not Corrected for Side Shear)

Lateral load = 1885 kips

Lateral load = 2194 kips

Lateral load = 2480 kips

Lateral load = 2806 kips
Lateral Displacements (Corrected for Side Shear)

Lateral load = 1885 kips

Lateral load = 2194 kips

Lateral load = 2480 kips

Lateral load = 2806 kips
Lateral Test# L5

P-Y Curves

Not corrected for side shear

Corrected for side shear
Lateral Displacements (Not Corrected for Side Shear)

Lateral load=1325 kips

Lateral load= 1880 kips

Lateral load=2420 kips

Lateral load=2845 kips
Lateral Displacements (Corrected for Side Shear)

Lateral load = 1325 kips

Lateral load = 2090 kips

Lateral load = 2420 kips

Lateral load = 2845 kips
Lateral Test# L6

P-Y Curves

Not corrected for side shear

Corrected for side shear
Lateral Displacements (Not Corrected for Side Shear)

<table>
<thead>
<tr>
<th>Lateral load</th>
<th>y (in)</th>
<th>Depth (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1330 kips</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2090 kips</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2450 kips</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2815 kips</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Lateral Displacements (Corrected for Side Shear)

Lateral load = 1330kips

Lateral load = 2090kips

Lateral load = 2450kips

Lateral load = 2815kips
Lateral Test# L7

P-Y Curves

---

**Not corrected for side shear**

---

**Corrected for side shear**

---
Lateral Displacements (Not Corrected for Side Shear)

Lateral load = 3056kips

Lateral load = 4000kips

Lateral load = 5035kips

Lateral load = 6000kips
Lateral Displacements (Corrected for Side Shear)

Lateral load = 3056kips

Lateral load = 4000kips

Lateral load = 5035kips

Lateral load = 6000kips
Lateral Test# L8

P-Y Curves

Not corrected for side shear

Corrected for side shear
Lateral Displacements (Not Corrected for Side Shear)

Lateral load = 2021 kips

Lateral load = 3069 kips

Lateral load = 4535 kips

Lateral load = 6035 kips
Lateral Displacements (Corrected for Side Shear)

Lateral load = 2021kips

Lateral load = 3069kips

Lateral load = 4535kips

Lateral load = 6035kips
Lateral Test# L9

P-Y Curves

Not corrected for side shear

Corrected for side shear
Lateral Displacements (Not Corrected for Side Shear)

- Lateral load = 1620 kips
- Lateral load = 2046 kips
- Lateral load = 2463 kips
- Lateral load = 2669 kips
Lateral Displacements (Corrected for Side Shear)

Lateral load = 1620kips

Lateral load = 2046kips

Lateral load = 2463kips

Lateral load = 2669kips
Lateral Test# L10

P-Y Curves

Not corrected for side shear

Corrected for side shear

Original P-Y curve
Modified P-Y curve

Original P-Y curve
Modified P-Y curve
Lateral Displacements (Not Corrected for Side Shear)

- Lateral load = 1812 kips
- Lateral load = 3236 kips
- Lateral load = 4062 kips
- Lateral load = 5035 kips

Depth (ft) vs. y(in) for different lateral loads and tests.
Lateral Displacements (Corrected for Side Shear)

Lateral load = 1812 kips

Lateral load = 3236 kips

Lateral load = 4062 kips

Lateral load = 5035 kips
Lateral Test# L11

P-Y Curves

**Not corrected for side shear**

![Graph showing P-Y curves without correction for side shear.]

**Corrected for side shear**

![Graph showing P-Y curves corrected for side shear.]

Legend:
- Original P-Y curve
- Modified P-Y curve
Lateral Displacements (Not Corrected for Side Shear)

Lateral load = 1899 kips

Lateral load = 2852 kips

Lateral load = 3976 kips

Lateral load = 5048 kips

Test

FB-Pier (modified p-y)

FB-Pier (original p-y)
Lateral Displacements (Corrected for Side Shear)

Lateral load = 1899 kips

Lateral load = 2852 kips

Lateral load = 3976 kips

Lateral load = 5048 kips
Lateral Test# L12

P-Y Curves

Not corrected for side shear

Corrected for side shear

Original P-Y curve
Modified P-Y curve
Lateral Displacements (Not Corrected for Side Shear)

Lateral load = 1380kips

Lateral load = 2050kips

Lateral load = 2450kips

Lateral load = 2760kips
Lateral Displacements (Corrected for Side Shear)

Lateral load = 1380kips

Lateral load = 2050kips

Lateral load = 2450kips

Lateral load = 2760kips
LIST OF REFERENCES


BIOGRAPHICAL SKETCH

Lila Dhar Niraula was born in Taplejung, Nepal, on July 12, 1969, and attended middle and high schools in the capital, Kathmandu. On completion of his high school (Advanced Level examinations of the University of Cambridge Local Examination Syndicate) in 1989, Lila joined Oxford University for a general engineering degree and graduated with a Master of Engineering in July 1994.

From January 1995, Lila worked as a mathematics teacher at Budhanilkantha High School in Kathmandu for nearly six years before joining the University of Florida as a graduate student in August 2001. He is expected to graduate in August 2004, with a master’s degree in civil engineering, specializing in geotechnical engineering.

Lila plans to work and practice geotechnical engineering in the Unites States for a period of time before returning to his home country, Nepal, to continue his professional career in engineering.