ABSTRACT

ROBINSON, BRENT ROSS. Models for Determining Effective Pile Lengths for Pile Bents. (Under the direction of Mohammad A. Gabr and Roy H. Borden).

Deep foundation supported bridge bents are cooperatively designed by geotechnical and structural engineers. The most robust analysis approach fully integrates both disciplines with nonlinear structural and geotechnical models. A simplified design approach is based on an elastic frame idealizing the deep foundations as fixed base columns. The equivalent length or depth to fixity can be chosen to match either the same design forces or deformations as the nonlinear models, but not both. For the most critical loads, this study proposes an approach to approximate the behavior of the nonlinear model with an elastic frame.

The models are developed with single pile lateral and axial analyses and compared to more refined analyses of the entire bent in FB-MultiPier, with SAP2000 as an independent verification tool, using pile sections with nonlinear soil, pile and pile cap material properties. The results for simple driven pile bents show that an equivalent frame model provides similar moment, shear, and displacement values as those obtained from both the SAP and MultiPier nonlinear analyses. Analysis results also indicated that the equivalent frame model parameters are particularly sensitive to the comparable selection of both axial and lateral loads. If lateral loads used to develop the equivalent model are higher than experienced, the axial and lateral deflections, and moments will also be higher. For design purposes this is conservative.

The results for simple drilled shaft bents indicated that the equivalent frame model provides responses that are comparable to those obtained from more rigorous finite element analyses. The study presents the results of the optimization of the support system by reducing the number, or size, of the shafts while maintaining acceptable level of safety.

Two models for driven pile toe load-displacement behavior are currently implemented in FB-MultiPier, a bridge design software package. The hyperbolic model implemented is primarily governed by the pile size, ultimate end bearing force as calculated by limit state methods or measured from load test programs and the soil's initial shear modulus. An empirical non-linear model is governed by the pile size and ultimate end bearing force. A review of the range of initial shear moduli measured in soils and correlated to common *in situ* tests indicates a typical range of around 1 to 50 ksi, depending on soil type and strength. Case histories from the literature are developed to compare the performance of the hyperbolic and nonlinear models. Application of the case history soil information to the respective toe models would indicate the hyperbolic model tends to under predict the stiffness behavior of the pile relative to the nonlinear model for the common range of shear modulus. This under prediction of pile toe stiffness can adversely affect the efficiency of the pile and cap design on predominantly end bearing piles supporting bridges.

Models for Determining Effective Pile Lengths for Pile Bents

by

Brent Ross Robinson

A dissertation submitted to the Graduate Faculty of
North Carolina State University
in partial fulfillment of the
requirements for the degree of
Doctor of Philosophy

Civil Engineering

Raleigh, North Carolina

2016

APPROVED BY:

Dr. Mohammed A. Gabr Committee Chair	Dr. Roy H. Borden Committee Co-Chair
Dr. Mervyn Kowalsky	Dr. M. Shamimur Rahman

DEDICATION

To Angie

BIOGRAPHY

Brent Robinson received his Bachelor of Science degree in Civil Engineering from Case Western Reserve University in Cleveland, Ohio in 1999. During his undergraduate degree, Brent undertook a co-op with Goble Rausche Likins and Associates, which later turned into a full time job after graduation. After many adventures on construction sites, in seminar lectures and at conferences in the United States and around the world, Brent returned for graduate work at North Carolina State University in 2004 and completed a Master of Science Degree in 2006. He started his Ph.D. program after that, working on the NCDOT undercut project as a Dwight D. Eisenhower Fellow through the US Department of Transportation.

He is currently a partner and vice president at GRL Engineers, Inc. and Pile Dynamics, Inc. where he continues to provide technical lectures, training, technical support and to drive the firms' continuing efforts to improve their testing methods for deep foundations. He is an active member of ASCE, DFI, PDCA, ADSC, ASTM and currently serves on two committees in the soil mechanics section of the Transportation Research Board.

Brent is lucky to be married to Angela Robinson, M.D., M.P.H. and to be the father of two daughters, Sonia and Julia.

ACKNOWLEDGMENTS

The author thankfully acknowledges the foresight of the North Carolina Department of Transportation's Geotechnical and Structural Divisions. Their continued support of research programs at the university level to answer practical and significant question should be commended much more than it is. They are a progressive Department who are leaders in the Southeast, within the Transportation Research Board and AASHTO. The author also acknowledges the Dwight D. Eisenhower Fellowship program for its generous support, and for that program's unspoken effort to better leverage state research dollars with additional student resources.

I am grateful to my mentors at North Carolina State: Drs. Mohammed Gabr, Roy Borden, Shamimur Rahman, and Mervyn Kowalsky. The efforts of Greg Lucier, Jerry Atkins and Dr. Sami Rizkalla at the Constructed Facilities Laboratory are also appreciated and remembered fondly. The Civil Engineering Department is an engaging and wonderful place to be and to study. I enjoyed collaborating on NCDOT research projects with Vinicio Suarez, Pablo Robalino, Young Jin Park, Ben Possiel, Aidcer Vidot, Ben Cote, and Sangchul Pyo.

I also must acknowledge my colleagues at Pile Dynamics and GRL Engineers. It is a stimulating, exciting and fulfilling place to practice engineering.

And to my family: Angie, Sonia and Julia.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1: Introduction and Background

Deep foundation supported bridge bents are cooperatively designed by geotechnical and structural engineers. As described in Hannigan et al. (2016), structural engineers suggest preliminary loads on the bent and tolerable deformations. Geotechnical engineers execute and evaluate the results of a subsurface exploration and laboratory testing program, suggesting potential foundation type. Once it is decided that deep foundations should be investigated, the geotechnical engineer evaluates one or more driven or drilled foundation geometry and performs axial and lateral design. This preliminary design is sent to the structural engineer, where refined modeling determines the number of piles to satisfy strength, extreme event, and service limit states per AASHTO (2014). At this point, the foundation design may go back to the geotechnical engineer to evaluate pile group effects and other installation considerations. Once the foundation is finalized, design of the pile cap and the remainder of the bridge design can proceed by the structural engineer.

A unified model containing both structural and geotechnical modeling elements in a software package, such as MultiPier (BSI, 2015) or SAP2000 (CSI, 2015) is the most robust approach to analyzing proposed designs. However, technological, historical, organizational or financial reasons may keep designers from using that approach. One alternative is to perform a frame analysis, with a simplified model for the foundation element.

The simplified design approach to bridge bents is based on determining an equivalent pile length (or point of fixity) along the deep foundation length and then considering the bents as free-standing elastic frames with columns fixed at their bases. In this approach, the soil is not explicitly modeled as a component of the bridge system. The depth to fixity is usually chosen so that the equivalent bent matches the stiffness of the real soil-bent system. It is also possible to define an equivalent model to develop the same design forces (i.e., moment, shear and axial force) in both the lateral pile analysis and in the frame analysis. However, it is not possible to define a point of fixity to match both design forces and the element's stiffness to vertical and lateral deformation at the same time. Despite its limitations, this approach is practical and well established because it produces a simple and inexpensive model for structural design. Once the point of fixity is estimated, the frame approach can be implemented in many available elastic analysis/design programs that do not allow for input of nonlinear springs to represent soil and foundation interaction.

When the AASHTO foundation design specifications for bridges using load resistance factor design (LRFD) were adopted in 2007, the bulk of the development effort for the geotechnical aspects of bridge design was to devise specifications for strength and extreme event limit states. These limit states and their resistance factors have been refined, revised and locally calibrated in the intervening years. The service limit states in the current LRFD specifications (AASHTO, 2014), or the calculation of deformations and foundation settlements, are largely unchanged in AASHTO design specifications from the allowable stress design methods.

A service limit state design framework was investigated by Modjeski and Masters, Inc. (2015) after the LRFD design specifications were adopted. Suggested revisions to the AASHTO LRFD Bridge Design specifications include evaluating foundation deformation for each foundation element based on methods approved by the owner and dependent on the foundation type. The lateral and vertical deformations are to be considered in the design and relative angular distortions are to be computed between each abutment or other substructure element, in a manner similar to those described by Wahls (1981).

As the service limit state is adopted, additional research will be needed to examine the accuracy of existing and newer analysis methods for calculating immediate and consolidation settlement of foundations. If an equivalent frame model is still to be used, the ability of that model to estimate vertical and lateral deformations similar to those calculated by more robust analysis methods must now also be considered.

PROBLEM STATEMENT AND ORGANIZATION

This study focuses on the approximation of full nonlinear bent models by a free standing frame model. The nonlinear models include soil-structure interaction via springs to model lateral and axial loads. The frame model idealizes the bent's behavior with equivalent column lengths of a foundation element with fixed head or pinned head connections at the bent cap. The implications for structural design of the pile and bent cap elements are reviewed, as are the ability of the equivalent models to predict transverse and longitudinal displacements for service limit state concerns. The approach is used for both driven and drilled pile foundations.

During the initial investigations for the driven pile bent, significant issues were observed. The deformations due to axial loads transferred to the pile tip were over predicted. For a primarily end bearing pile, these large deformations yielded large differential displacements, which yielded large moments in the bent cap. The study also investigates the hyperbolic driven pile end bearing-displacement models used in FB-MultiPier (BSI, 2015) to check what types of soil modulus parameters might be needed to yield reasonable comparisons to a series of load tests reported in the literature.

This document is organized in three parts: i) comparing finite element and elastic frame models to calibrate simplified lateral analyses of bridge bents supported by driven piles, ii) extending the simplified lateral analyses to bridge bents supported by drilled shafts and iii) investigating the driven pile toe resistance vs displacement model assumed in the finite element studies and its effects on the results of part i).

The section entitled Simplified Lateral Analysis of Deep Foundation Supported Bridge Bents: Driven Pile Case Studies introduces the equivalent length models for pinned and fixed head connections. It compares the results of fixed base frame analyses using the equivalent length models to fully nonlinear models of pile bents with soil structure-interaction modeled with interface p-y or t-z springs on case studies for a variety of typical driven pile bent designs supplied by the North Carolina Department of Transportation. It reviews the sensitivity of the equivalent model parameters for a range of axial loads, and considers the relative differences in displacements, bent cap moments and axial pile loads for the nonlinear and equivalent frame models for each bridge modeled.

The section entitled Configuration Optimization of Drilled Shafts Supporting Bridge Structures: Three Case Studies uses the equivalent length models to compare nonlinear finite element models with the equivalent free standing frame model. The deep foundation elements used are larger diameter drilled shaft elements with lateral and vertical axial load-deformation relationships specially formulated for drilled shafts as described in FB-MultPier. The controlling moments, axial and shear forces are compared for the nonlinear and equivalent frame models, and additional models are reviewed to optimize the foundation design of each of the three bridges in the case study.

The final section, entitled Pile Toe Displacement Models for Driven Pile Bent Design in FB-MultiPier reviews the hyperbolic end bearing-axial displacement model of McVay et al. (1989) and the model from the American Petroleum Institute RP2A (2003). The primary parameter for determining the stiffness of the hyperbolic model is the initial shear modulus, and a range of models from the literature are reviewed to determine typical boundaries for this parameter in soils. These shear moduli and the two models are then evaluated by comparing the models' results to results reported in the literature. The effect on the cap moments for a bent cap supported primarily by end bearing is evaluated.

REFERENCES

American Association of State Highway and Transportation Officials (AASHTO). (2014). AASHTO LRFD Bridge Design Specifications, US Customary Units, Seventh Edition, with 2015 Interim Revisions. American Association of State Highway and Transportation Officials, Washington, DC, 1960 p.

Bridge Software Institute (BSI). (2014). *FB-MultiPier Version 4.19.3 Manual*,. University o Florida.

Computers and Structures, Inc. (CSI). (2015). SAP2000.

Hannigan, P., Rausche, F., Likins, G., Robinson, B., and Becker, M. (2106, forthcoming). *Geotechnical En*gineering Circular No. 12 –Volume I. Design and Construction of Driven Pile Foundations. FHWA-NHI-16-009.

McVay, M.C., Townsend, F.C., Bloomquist, D.G. and Caliendo, J.A. (1989). "Numerical Analysis of Vertically Loaded Pile Groups." Foundation Engineering: Current Principles and Practices. *Proceedings of the Congress. Edited by F. H. Kulhawy, American Society of Civil Engineers.* 675-690.

Modjeski and Masters, Inc.. (2015). Bridges for Service Life Beyond 100 Years: Service Limit State Design. SHRP-2 Report S2-R19B-RW-1. Transportation Research Board, Washington D.C. 521 pages.

Wahls, H. (1981). “Tolerable Settlements in Buildings” Proceedings of the Geotechnical Engineering Division, ASCE, Vol. 107, GT11, 1489-1504.

CHAPTER 2: Simplified Lateral Analysis of Deep Foundation Supported Bridge Bents: Driven Pile Case Studies

Brent Robinson, P.E., M.ASCE,, Vinicio Suarez, Mohammed A. Gabr, P.E., F.ASCE, and Mervyn Kowalsky, P.E., M.ASCE

Previously published as: Robinson, B., Suarez, V. Gabr, M. and Kowalsky M. (2011). "Simplified Lateral Analysis of Deep Foundation Supported Bridge Bents: Driven Piles Case Studies." ASCE Journal of Bridge Engineering, V 16, n 4.

ABSTRACT

A simplified approach for modeling soil and foundation system supported bridge bents is applied to three bridges that represent three pile types and three superstructures. This *point of fixity* approach is applied by modeling the bridge bent substructure as an elastic frame. The models are compared to more refined analyses in FB-MultiPier, with SAP as an independent verification tool, using pile sections with nonlinear soil, pile and pile cap material properties. The results for simple pile bents show that an equivalent frame model provides similar moment, shear, and displacement values as those obtained from both the SAP and MultiPier nonlinear analyses. Analysis results also indicated that the equivalent frame model parameters are particularly sensitive to the comparable selection of both axial and lateral loads. If lateral loads used to develop the equivalent model are higher than experienced, the axial and lateral deflections, and moments will also be higher. For design purposes this is conservative.

C.E. Database: Bridge design, Model verification, Numerical Analysis, Numerical Models , Pile Structures,

INTRODUCTION

In practice, deep foundation supported bridge bents are often designed by structural and geotechnical engineers in tandem. Geotechnical engineers suggest pile sizes and lengths for given sets of axial and lateral loads. Structural engineers determine the design loads according to code (e.g., AASHTO dead, live, wind, impact loading, etc.) and conduct the structural design of the piles, bent caps and remainder of the bridge structure.

In general, a simplified design approach to bridge bents is based on determining a point of fixity along the deep foundation length and then considering the bents as free-standing elastic frames with columns fixed at their bases. In this approach, the soil is not explicitly modeled as a component of the bridge system. The depth to fixity is usually chosen so that the equivalent bent matches the stiffness of the real soil-bent system. It is also possible to define an equivalent model to develop the same design forces (i.e., moment, shear and axial force) in both the lateral pile analysis and in the frame analysis. However, it is not possible to define a point of fixity to match both design forces and stiffness at the same time. Despite its limitations, this approach is practical and well established because it produces a simple and inexpensive model for structural design. Once the point of fixity is estimated, the frame approach can be implemented in many available elastic analysis/design programs that do not allow for input of nonlinear springs that represent soil and foundation interaction.

Point of Fixity Approach

Commonly-used point of fixity equations were proposed by Davisson and Robinson (1965) and have been incorporated into AASHTO (2007). However, in this case their use is recommended for the assessment of buckling effective length only of the columns that constitute the frame members. These equations are based on beam-on-elastic foundation theory and are set such that the results from the point of fixity model can approximately match bending and buckling responses simultaneously. In this case, the soil is assumed to be perfectly elastic, which is rarely the case. These point of fixity models do not distinguish between free- and fixed-headed piles and, therefore, do not yield accurate lateral displacements for the pile bent when used in a frame analysis.

Another approach to estimate point of fixity utilizes the results of a single pile lateral analysis (NCDOT 2005). Some engineers have used the point of maximum negative moment or the point of maximum negative displacement along the pile length to define a point of fixity for the purpose of performing elastic frame analysis of the bridge structure system. Although no universal agreement exists as to the definition of the location of the point of fixity, such a definition nonetheless will have an impact on the computed stresses and displacements of a bridge structure. There is no single definition for determining a point of fixity; however, there is general agreement that a deeper point of fixity leads to a more conservative design.

Robinson et al. (2006) observed that analysis of a cantilevered column with an *equivalent* length determined by the point of fixity does not yield results that match the magnitudes of maximum moments, lateral pile top displacements, or buckling behavior obtained from nonlinear single pile lateral analysis. This observation led to the formulation of an equivalent model for pile bents in which the point of fixity was set to match the pile response due to design forces. The stiffness is also matched by application of a factor to the moment of inertia of the pile section. The model is presented for piles with pinned head and fixed head conditions in Figure 1.

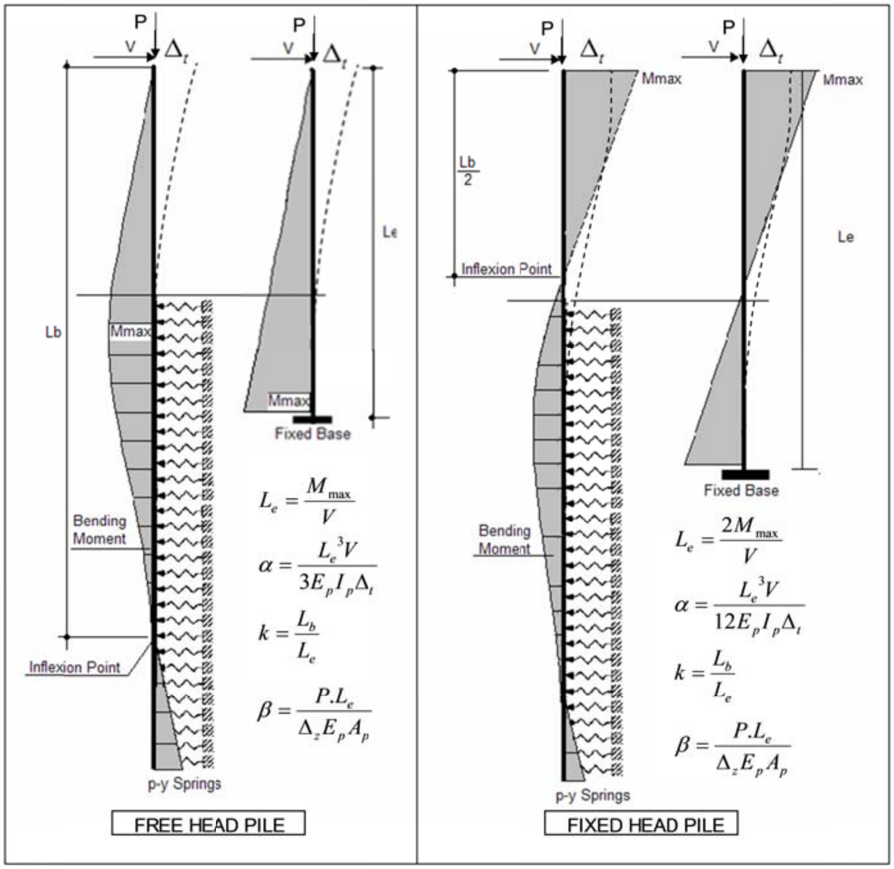

Figure 1. Equivalent model parameters (Robinson et al 2006)

To model a pinned head condition, an equivalent column model is presented by Robinson et al. (2006) as follows:

$$L_e = \frac{M_{\max}}{V} \tag{1}$$

$$\alpha = \frac{L_e^3 V}{3E_p I_p \Delta_t} \tag{2}$$

$$k = \frac{L_b}{L_e} \tag{3}$$

For the fixed head condition, the equivalent model parameters can be evaluated from

$$L_e = \frac{2M_{\max}}{V} \tag{4}$$

$$\alpha = \frac{L_e^{\,3} V}{12 E_p I_p \Delta_t} \tag{5}$$

$$k = \frac{L_b}{L_e} \tag{6}$$

And for both the free head and fixed head conditions,

$$\beta = \frac{P.L_e}{\Delta_z E_p A_p}, \tag{7}$$

where:

L_e Length of a pile fixed at the base that will develop the same maximum moment, M_{max}, as in the nonlinear soil-pile model under the application of the lateral load, V, at the top.

M_{max} Maximum moment developed in both the equivalent model and the nonlinear soil-pile model.

V Lateral force applied at the top of the pile in both the equivalent model and the nonlinear soil-pile model.

α Inertia reduction factor that when multiplied by the inertia of the pile, I_p, in the equivalent model will give the same lateral stiffness (i.e., will result in the same lateral displacement) of the nonlinear soil-pile model.

E_p Elastic modulus of the pile material.

I_p Moment of inertia of the pile about the axis perpendicular to the applied load.

Δ_t Lateral displacement at the top of the pile caused by the applied lateral force, V.

k Factor that when multiplied by the equivalent length, L_e, yields the effective length for a stability (buckling) check of the pile.

L_b Effective length for a stability (buckling) check of the pile, taken from the moment diagram in the nonlinear soil-pile model between the top of the pile and the first point of zero moment (inflection point).

b Factor applied to the area of the pile in the equivalent model in order to result in the same axial deformation as the nonlinear soil-pile model under the axial load, P.

A_p Area of the pile section.

P Applied axial load.

Δ_z Axial displacement at the top of the pile.

The proposed method of calculating a point of fixity uses the moments and displacements estimated from a single pile lateral analysis, such as those moments and displacements available from LPILE (Ensoft, 2004) or FB-MultiPier (BSI, 2004). To capture the disparate behavior in the transverse and lateral directions, the single pile analysis can be run twice, once with a free head condition (longitudinal direction) and once with a head condition fixed against rotation (transverse direction). This approach, however, leads to two points of fixity that can be modeled by either adding a restraining spring in the shallow direction, or using the deeper value for an analysis that would tend to result in higher design moments and shear forces in the pile.

For this study, a series of three bridge structures were selected and analyzed to establish a baseline description of the current state of practice and study the applicability of the above point of fixity model. The selected case studies are reviewed, and soil and structural element information is extracted for use in detailed 3-D numerical analyses. Modeling of the bridge structures is performed within the framework of the FB-MultiPier suite of programs for both structural and geotechnical analyses. In addition, SAP 2000 (CSI, 2003) is used in the modeling to validate the results of FB-MultiPier and to run the simplified fixed base, elastic frame analyses. Results from both programs are compared with those obtained from the point of fixity approach. In addition, issues related to conservatism and the impact of current design assumptions are discussed.

3-D MODELING SOFTWARE: PARAMETERS AND MODEL VALIDATION

Evaluating the point of fixity model requires more refined models for comparison. To show results from a nonlinear analysis approach are reasonably repeatable, two separate software packages were selected. FB-MultiPier was selected as the primary modeling tool for analysis because it: (1) has an interactive bridge bent software wizard built in; (2) automatically models the soil resistance (lateral and axial, single and group) using methods that represent the current state of practice; (3) allows typical linear or nonlinear bent cap models to be saved and utilized; and (4) has the option of modeling bridges with multiple bents connected by idealized superstructure elements. FB-MultiPier requires the input of soil parameters, pile type and length, bent cap dimensions and geometry, and AASHTO load combinations. The program includes a wind load generator for wind loading conditions, but is limited to only nine cases of a particular loading type for a single run.

In addition to the use of FB-MultiPier, SAP 2000 Version 8.2.5 was used for comparison analyses. Using SAP 2000, models for each bent of the seven case studies are created, both for point of fixity-type frame analysis and to model pile-soil responses using full-length piles with nonlinear springs along the length.

Practical Limitations of Both Programs

FB-MultiPier is a nonlinear analysis program that has been specialized in soil-structure interaction using P-y models. The program automatically generates the P-y springs for a wide range of soil models. This tool is best suited for the assessment of structures that have been previously designed.

SAP 2000 is a general analysis and design program, capable of performing elastic and nonlinear analysis of multiple types of structures. The program supports nonlinear springs to which a P-y response can be assigned. However, because it is not specialized in soil-structure interaction, there is no automatic generation of P-y curves, and, therefore, the model springs must be input manually.

Both FB-MultiPier and SAP can be used as analysis tools, but both lack design modules to fulfill AASHTO requirements. For use as analysis tools, a bridge that has been preliminarily designed is input into these programs to find displacements, forces, moments, and other design quantities. For use as design tools, both programs would require additional pre- and post-processing. Wind loads can be generated in FB-MultiPier but not in SAP, and for both programs, the live loads, dead loads, traction forces, and other load cases must be developed outside the program. The results from either program include resultant forces and moments in the piles/shafts and bent cap, displaced shapes of the structure and its system components and, in FB-MultiPier only, demand/capacity ratios (DCRs) for any section modeled as a nonlinear system. The moments, forces, and displacements in the foundation elements or bent caps can be used to determine whether the ultimate capacity of a particular member is adequate for a predicted amount of movement. Once these values are calculated, the bent cap rebar can be verified or determined using design equations.

Soil Modeling

The built-in soil models in FB-MultiPier are used to model the interaction between the soil and foundation elements (piles or shafts). For lateral analysis, the P-y approach is used, and the t-z approach is used for the axial analysis of the foundation elements.

Lateral (P-y) Model

Lateral pile load deformation analysis is performed by modeling the foundation elements as a series of elastic frame elements, and the soil as a series of nonlinear springs. The lateral P-y models used for this study were Reese et al. (1974)'s model for sands, Matlock (1970)'s model for soft clay below the water table and Reese et al. (1975)'s model for stiff clay below the water table.

Once the model parameters are input and the P-y curves are developed in FB-MultiPier, approximations of those curves are transferred to the nonlinear SAP model. Because SAP 8.2.5 has no built-in P-y soil models, those generated by and displayed in FB-MultiPier's soil input section are used. To transfer to SAP, 20 data points calculated by FB-MultiPier are copied as load-deformation definitions for nonlinear springs.

FB-MultiPier presents tabular results for both the top and the bottom of a selected soil layer. It assigns a lateral soil spring load displacement curve for each node. Any interpolation between the model parameters at the top and bottom of the layer in FB-MultiPier is linear. In SAP, however, the P-y curves from the top and the bottom of the layer are used as starting points for linear interpolation of the P-y curves between the two depths.

While not perfect, this process resulted in values in SAP as close as could be expected to those actually used in FB-MultiPier. This use of the FB-MultiPier displayed P-y curves ultimately led to some of the differences in the transverse and longitudinal displacement observed from both programs. Specifically, two major differences arose:

i) FB-MultiPier presents tabular values in 20 equal displacement increments. This incremental spacing usually means insufficient resolution for the initial, highly nonlinear portion of the P-y curve. For small displacements, therefore, the lateral soil reaction force in SAP is generally lower than that actually predicted by the FB-MultiPier analysis. This lower force indicates that the system is less stiff, which means SAP produces higher displacements.

ii) In the case of soft clays below the water table, the ultimate lateral resistance, P_{ult}, increases in magnitude bilinearly with depth. The SAP approximation, determined from the ultimate capacities at the top and bottom of the layer, increases linearly with depth. Both ultimate lateral capacity curves are shown in Figure 2. SAP's linear increase leads to lower maximum forces compared to FB-MultiPier, and therefore underpredicts the stiffness and overpredicts the displacements.

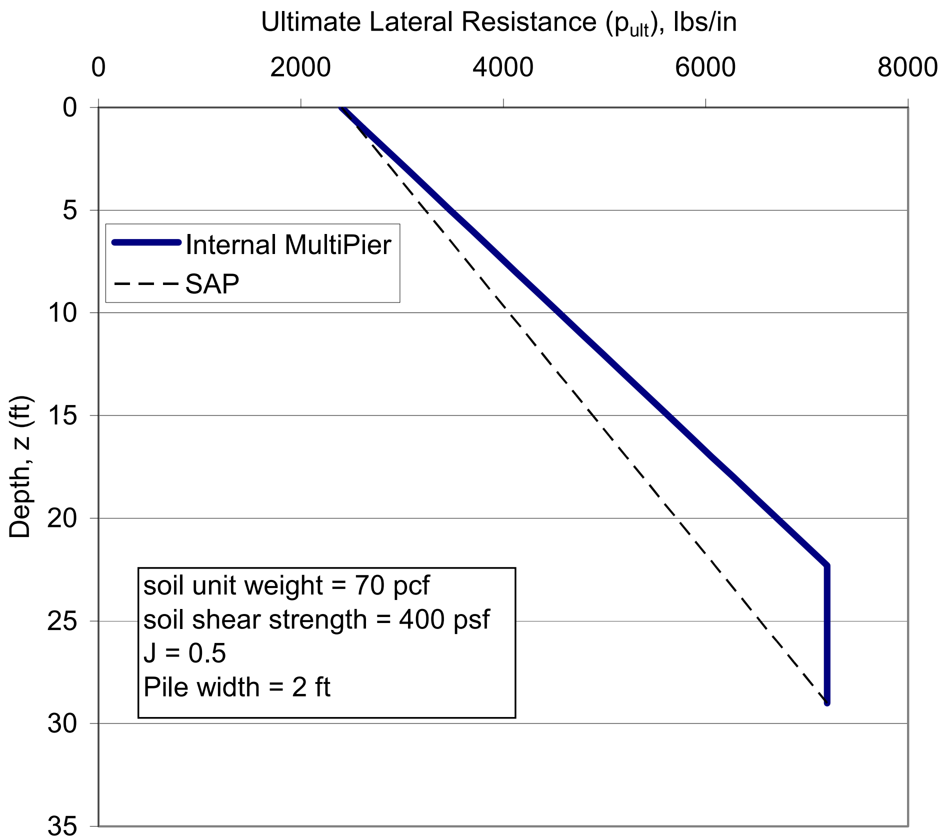

Figure 2. Variation between SAP and FB-MultiPier maximum lateral stiffness with depth for development of the Matlock (1970) P-y curves.

Axial (t-z), Skin Friction

To predict the load-deformation response in the axial direction, FB-MultiPier uses a method similar to the P-y analysis. For the axial response, the pile is again modeled as a series of frame elements, and the soil is modeled as a series of nonlinear springs along the side of the pile to model shaft resistance and at the pile's toe to model end bearing. Similar to P-y analyses, the spring stiffness for a particular soil element is a function of depth as well as the soil's shear strength and stiffness parameters. Unlike P-y curves, which calculate the spring stiffness versus lateral displacement, the t-z curves for the axial springs are used directly to calculate the full force-deformation response. To calculate the force, the shear resistance mobilized t is multiplied by the circumferential area (circumference times length) of the foundation element.

FB-MultiPier uses an asymptotic shaft resistance model proposed by McVay et al. 1989. A typical plot of unit shaft resistance versus displacement is shown in Figure 3, along with a conceptual model of a discretized pile with springs modeling shaft and toe resistance. The initial stiffness of the curve is dictated by the low strain shear modulus, whereas the asymptotic behavior is determined as a function of shear strength.

Similar to the P-y curves, the t-z curves at the top and bottom of each soil layer can be viewed in FB-MultiPier. A table of data can be displayed, and these data are then copied and transferred to SAP. The shear stress values are multiplied by the pile perimeter and the length of the element used at each point in the model. In both SAP and FB-MultiPier, the increase in resistance from the top to the bottom of each layer is assumed to be linear.

Also, similar to the P-y curves, the 20 data points displayed in FB-MultiPier consider equally spaced displacements. Accordingly, the resolution of the early nonlinear portion of the curve is often coarse, as shown in Figure 3. Thus, the shaft resistance into the SAP model is often less stiff than that used in FB-MultiPier.

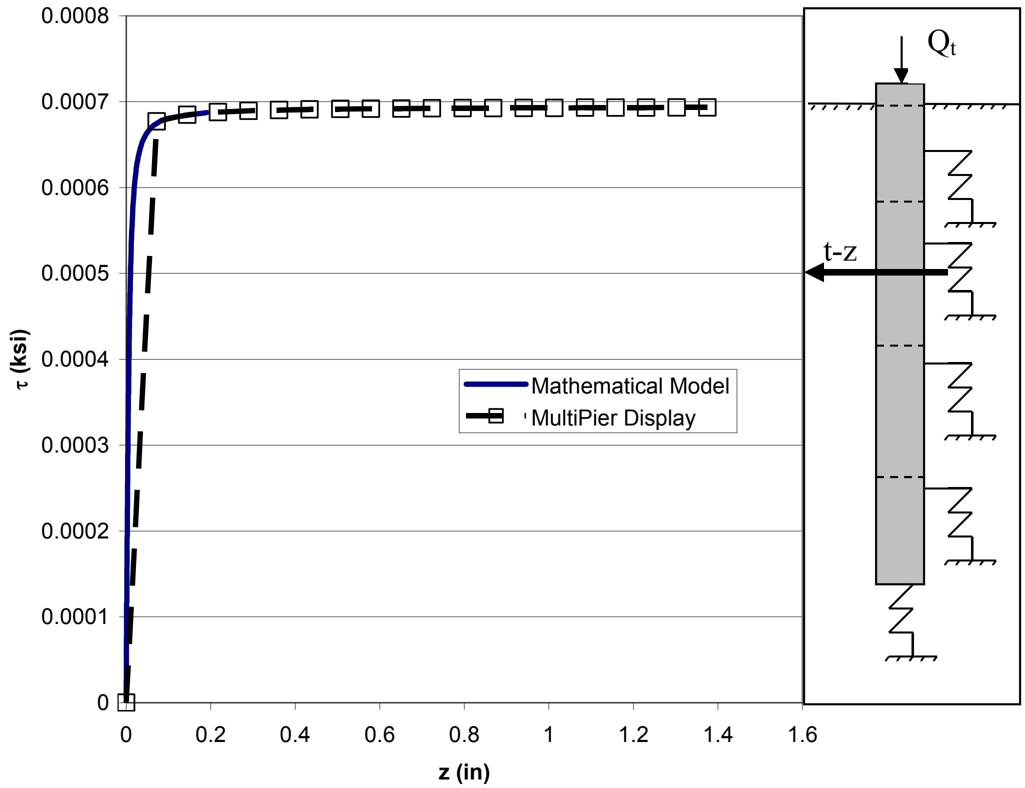

Figure 3. Example shear stress vs. displacement (t-z) curve (based on McVay et al., 1989) as applied to a soil spring, 18-inch pipe pile in sand, G_i =3.5 ksi.

Axial (q-z), End Bearing

The FB-MultiPier end bearing model was developed by McVay et al. (1989). The mathematical expression is shown in Equation 11.

$$z = \frac{Q_b(1-\nu)}{4r_oG_i\left[1-\frac{Q_b}{Q_f}\right]^2} \tag{11}$$

where:

Q_b = End bearing force applied to toe

Q_f = Ultimate end bearing force as calculated by limit state methods.

z = Axial displacement.

G_i = Initial (low strain) shear modulus.

r_o = pile radius

ν = Poisson's ratio

As can be seen in Figure 4, the FB-MultiPier toe response is very sensitive to the value of initial shear modulus input. In Figure 4, an ultimate toe resistance, Q_f, of 640 kips is input based on a limit state analysis such as Vesic (1977). In most load test interpretation methods, *total* pile capacity, Q_t, is sometimes defined as the capacity mobilized at displacements equivalent to 1 inch or 10% of the pile diameter or width. Most static toe resistance prediction models tend to give ultimate resistances that are mobilized at displacements slightly higher than expected to be experienced by the structure.

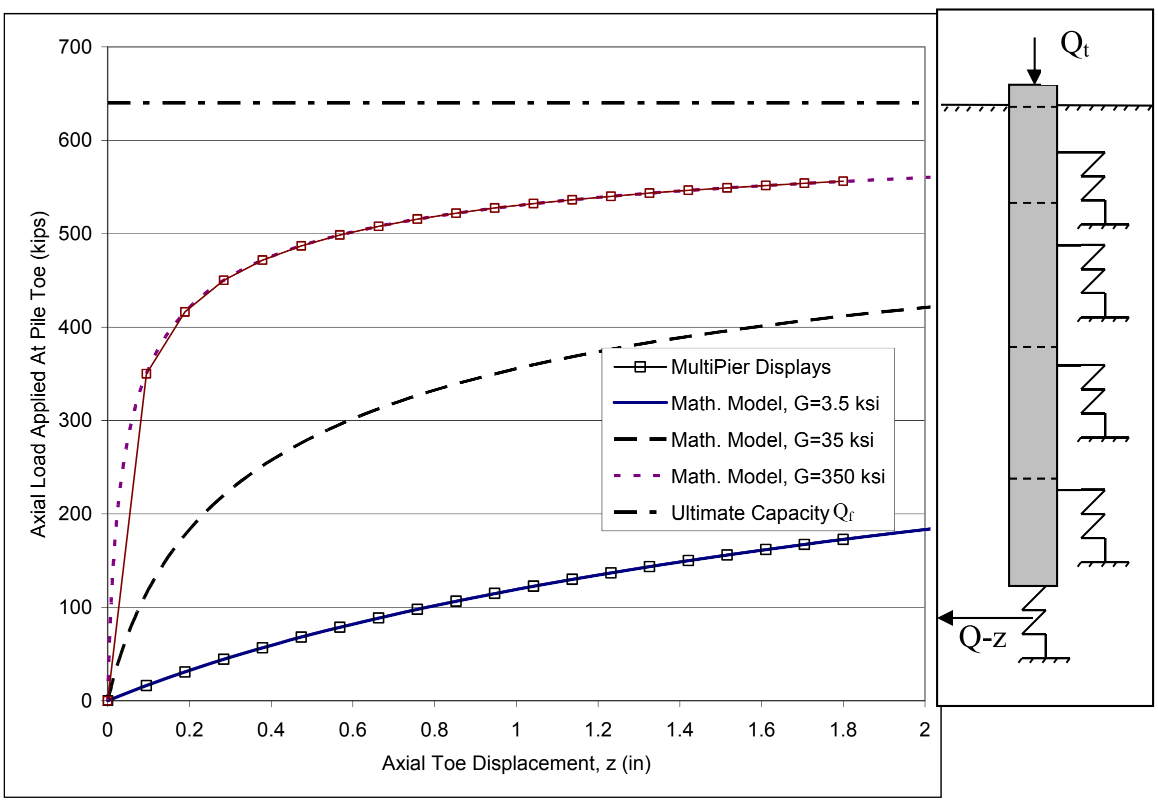

Figure 4. FB-MultiPier load-displacement (Q-z) curves for the model spring at the toe, using various shear moduli.

Using the standard static resistance model parameter recommendations in FB-MultiPier creates a situation in which reaching the ultimate end bearing capacity requires very large displacements. As shown in Figure 4, to obtain 90% of the 640 kip ultimate capacity on an

18-inch diameter pipe with a secant shear modulus of 3.5 ksi (a value typical of soft clay, according to BSI, 2004), the model predicts that the pile must move more than 300 inches. At a shear modulus of 35 ksi (a value typical for dense sand), the pile must move 30 inches. Even at a very high initial soil shear modulus of 350 ksi (shown in Figure 4 for illustrative purposes), the pile must move 3 inches to mobilize 90% of this example's ultimate capacity.

The FB-MultiPier toe model must be reviewed very carefully from a geotechnical standpoint. It was observed that high relative vertical displacements between two deep foundation elements within a single bent generate large moments in the bent cap, particularly under the predominantly live load cases. Vertical differential settlement is not likely to occur in the field because the live load is applied relatively quickly and equally among the bridge lanes (as opposed to a factored static load in the modeling), and the foundation elements are generally installed to relatively high blow counts layers. Accordingly, it was decided to "fix" the pile toe throughout the analysis presented in this paper. For this case, a model was adopted using a load-displacement curve that increases linearly until a displacement of 0.1 inch is reached; then plastic failure occurs at 1000 kips. These values are based on the need to limit the axial movement of the foundation elements.

Shaft and Toe Model Input Parameters: Initial Shear Modulus

The FB-MultiPier manual (BSI, 2004) suggests estimating an elastic modulus from the N-value, then calculating a shear modulus based on Poisson's ratio, while McVay et al. (1989) also use a correlation between the shear modulus and Cone Penetrometer Test (CPT) tip resistance, after Robertson and Campanella (1984). Figure 5 shows the estimated shear modulus obtained from these sources. This figure assumes a Poisson's ratio of 0.3 and the correlation of the SPT N-value to cone penetration resistance from Robertson et al. 1983 and shows estimations for the shear moduli of sands as between 1 and 10 ksi.

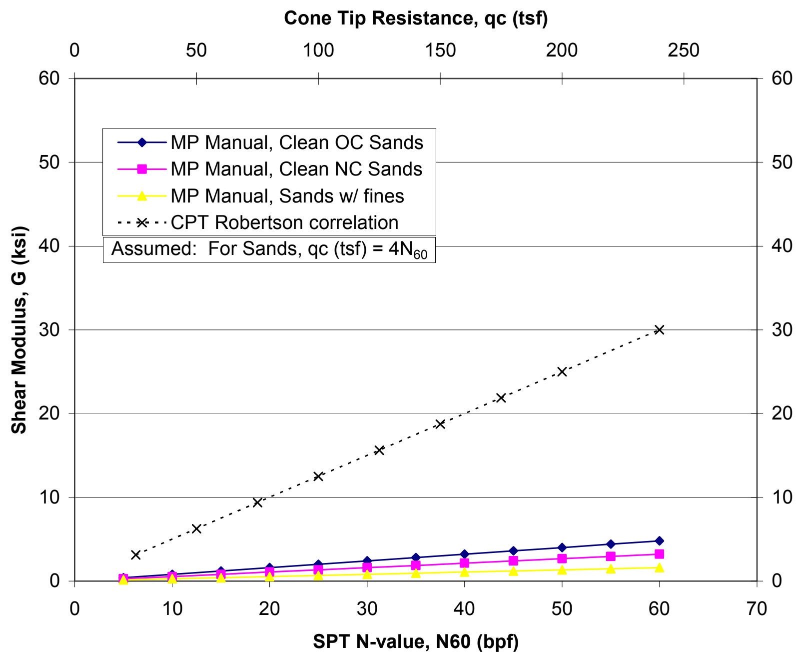

Figure 5. Shear Modulus Correlations suggested by BSI (2004).

As shown in the sample toe resistance curves in Figure 4, shear modulus values of around 3.5 ksi lead to a very soft toe response. The values obtained from Figure 5 do not seem to be low strain shear modulus values. Because the shear modulus determines the initial stiffness of the toe response, other approaches for evaluating low strain shear modulus are examined.

Figure 6 (Harden and Black 1968) and Figure 7 (Borden and Shao 1995) show shear modulus measurements from resonant column and torsional shear tests, which are small strain approaches. For the bridge bent t-z analysis presented in this paper, the initial shear modulus values presented in Figure 6 and Figure 7 are used, although it appears that Robertson and Campanella (1984)'s CPT correlation provides reasonable values as well.

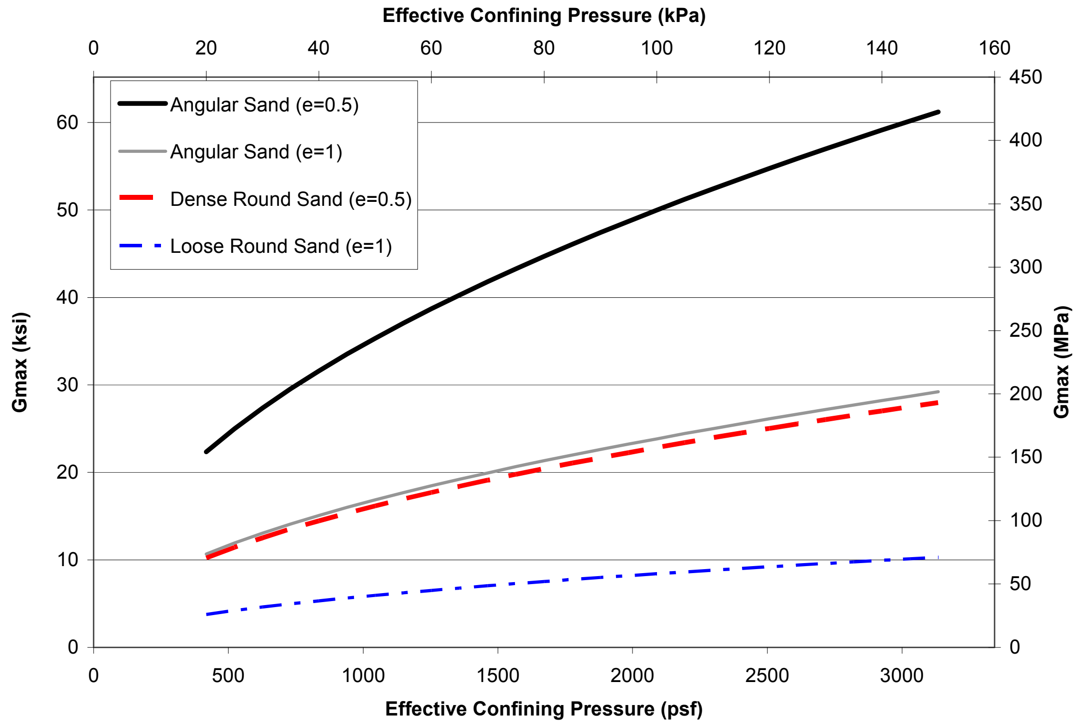

Figure 6. Initial (low strain) shear modulus values for sands (Harden and Black 1968).

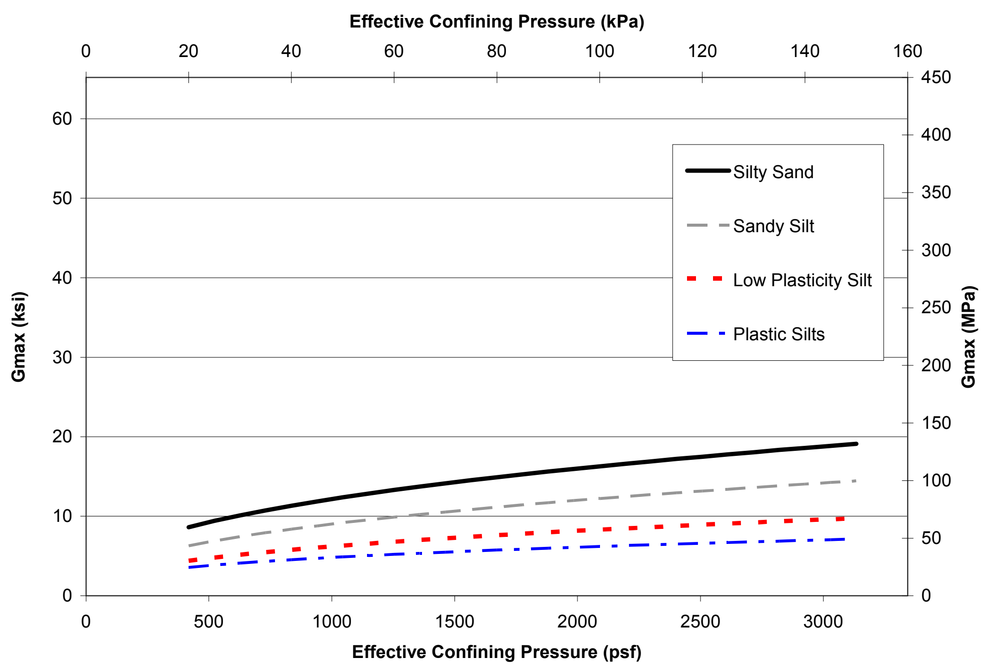

Figure 7. Initial (low strain) shear modulus for North Carolina residual soils (Borden and Shao 1995).

Subgrade Modulus

Calculation of the P-y curves requires the estimation of the subgrade modulus, k, for the top and bottom of each soil layer. The analysis approach presented herein uses the k-values recommended in the FB-MultiPier manual (BSI, 2004).

AASHTO LOAD CASES AND OUTPUT VALUES FOR DESIGN

The allowed number of load cases in FB-MultiPier includes one dead load case, up to five wind load cases, and up to nine live load cases, longitudinal forces, impact forces, or other cases. FB-MultiPier also includes a built-in wind load generator. SAP has no such limitations on the number of load cases, but the load cases are not built in. In this paper, AASHTO loading groups I, IA, (Live Load) II (Wind load) and III (Live load and braking load) were used in the analysis. The point of fixity model results and the FB-MultiPier results were synthesized for comparison to the nonlinear SAP results, according to the following steps:

a. Examine the maximum foundation top lateral displacements in the transverse, longitudinal, and axial directions, the maximum moment in the bent cap, and the maximum axial force, moment and DCR in the foundation elements.
b. For each maxima, identify the AASHTO load case and (if applicable) pile number associated with the maxima.
c. Using SAP, check the same load case from FB-MultiPier for the desired maxima and displacement, moment or force envelope.
d. Compare the output results from the three approaches.

For both the FB-MultiPier and SAP analyses, the concrete bent caps were modeled using linear-elastic frame elements. Because the amount of reinforcement for the “already designed” bridges was known, a cracked moment of inertia was estimated. Once this cracked moment of inertia was entered into both SAP and FB-MultiPier, the maximum moments could be identified and the required amount of rebar calculated.

The axial pile forces and moments are used in FB-MultiPier, along with the nonlinear pile section properties, to calculate a failure ratio or a DCR. As discussed in the FB-MultiPier manual (BSI 2004), the DCR “is an estimate of the percentage of the cross sections' capacity that has been reached for that particular loading state [due a particular load case].” A DCR of 1.0 or greater implies that the combination of moment and axial force falls outside of the moment-axial nominal interaction capacity curve. A DCR of less than 1 implies the section is able to sustain the range of AASHTO factored loading cases entered for the analysis.

CASE STUDIES

Three North Carolina bridge case studies were selected that use driven piles as the bridge foundation. Table 1 summarizes the location, type of superstructure, type of foundation, and dimensions and reinforcement details of the cap beams. In these analyses, the transverse direction is parallel to the bridge’s cap beam, and the longitudinal direction is perpendicular to both the bridge’s cap beam and the axis of the deep foundation. The axial direction is perpendicular to the cap beam and parallel to the axis of the vertical deep foundation.

ROBESON COUNTY BRIDGE

The Robeson County bridge spans the Lumber River on State Route (SR) 1303. It is a two-span bridge, and the single interior pile bent consists of H-piles with a concrete cap. The abutments are H-pile supported with wing walls. The superstructure consists of 15 3 ft x 1.75 ft prestressed concrete (PSC) cored slab units. For analysis of this bridge case study, the interior bent was modeled. The interior bent has 8 HP 14 x 73 piles. Each pile has a flexural stiffness *EI* of 7,569,000 kip-in^2 in the transverse direction, and a flexural stiffness EI of 21,141,000 kip-in^2 in the longitudinal direction. The total length of the piles is 55 feet. The anticipated factored axial dead load per pile is 60 kips. The anticipated factored lateral load acting in the transverse direction is 2 kips per pile, and the anticipated factored lateral load acting in the longitudinal direction is 1.7 kips per pile.

Lateral group analysis considers the spacing between the piles, which for this bridge is 72 in., or slightly greater than 5 times the 14-in. width of the pile (5D). From the FB-MultiPier manual (BSI, 2004), the leading pile’s P-y multiplier is 1; the adjacent pile’s multiplier is 0.85; and the multiplier for all the other piles is 0.7. For the 5D spacing, the axial group capacity was considered to be unaffected.

Geotechnical Characteristics

The soil boring shows the groundwater level to be at the surface. From a depth of up to 3 ft below the ground surface, very loose silty sand was reported. The SPT N-value for this layer was 1 blow per foot. Next, fine to coarse sand with gravel was encountered from 3 to 12 ft, with N values averaging 21 blows per foot. Fine to coarse sand was reported from 12 to 22 ft, with N values ranging from 4 to 15 blows per foot.

From 22 to 43 ft, the coarse sands of the Black Creek formation were encountered. N-values were generally in the teens through this layer. Very stiff silty clay material was encountered from 43 to 53 ft with very high N-values of 62 and 70 blows per foot. The boring terminated 66 ft below the ground surface at dense fine to coarse sands. As designed, the piles terminated in the 62 to 70 blow count silty clay material.

Point of Fixity

Results from the nonlinear single pile analysis, as well as the input data for the calculations of the equivalent model parameters described in Equations 1 through 7, are shown in Figure 8 and Figure 9 for a fixed head and free head condition, respectively. The equivalent model parameters were calculated and are shown in Table 2. Figure 8 and Figure 9 also show the location of the point of maximum negative moment and point of maximum negative deflection. These points, as indicated previously, have been commonly defined as the point of fixity in practice.

Table 1. List and Description of Cases Studies: Locations and Construction Details

No.	Bridge Name and Location	Bents and Foundation		Bent Cap Dimensions and Reinforcement		Bent Super/Substructure Connection	
		Interior Bents and Foundation Type	**End Bents and Foundation Type**	**Interior Bents**	**End Bents**	**Interior Bents**	**End Bents**
1	Robeson County Bridge, 2 spans (30 and 40 feet)	One bent, Eight HP 14 x 73, 55 ft long; 5 ft unsupported (without scour); 8 ft (with scour), end piles battered 1:8	Two bents, Eight HP 12 x 53, 55 ft long; Four brace piles battered 1:4	36 inch wide by 30 inch deep Class A concrete beam, Five #9 bars (top) and four #10 bars (bot.)	33 inch wide by 30 inch deep (minimum) Class A concrete beam with wing walls	Two rows of 15 elastomeric bearing pads, ¾ in. thick, Type II (Expansion Joints)	One row of 15 elastomeric bearing pads, ¾ in. thick, Type I (Fixed Joints)
2	Northampton County Bridge, 3 spans (61, 120 and 49 feet)	Two bents, Five 24-inch diameter closed end steel pipe piles, 0.5 inch thick wall; 60 ft long with ~7 to 8 feet unsupported	Two bents, Seven HP12x53; 70 ft long; Four brace piles battered 1:4	50 inch wide by 39 inch deep Class A concrete beam, Six #9 bars (top and bottom)	30 inch wide by 30 inch deep (minimum) Class A concrete beam with wing walls	Two rows of four elastomeric bearing pads (Type II for 61 and 49 ft spans, 2-7/16 inch thick; Type IV for 120 ft span, 3-5/16 inch thick)	One row of four elastomeric bearing pads (Type II, 2-7/16 inch thick)

Table 1 Continued

No.	Bridge Name and Location	Bents and Foundation		Bent Cap Dimensions and Reinforcement		Bent Super/Substructure Connection	
		Interior Bents and Foundation Type	End Bents and Foundation Type	Interior Bents	End Bents	Interior Bents	End Bents
3	Halifax County Bridge, 9 spans (35, 40 and 50 feet)	Eight bents, eight 18-inch square prestressed, precast concrete piles; 40 to 50 ft long with 8 ft unsupported; Bent 1 is a 1:8 battered A-frame, but all other piles are vertical.	Two bents, Seven HP12x53; 50 ft long; Three brace piles battered 1:4	39 inch wide by 30 inch deep Class A concrete beam, Five #9 bars (top) and four #9 bars (bottom)	33 inch wide by 30 inch deep Class A concrete, Five #9 bars (top) and four #9 bars (bottom) beam with wing walls	Two rows of 15 elastomeric bearing pads (Type I and II, 1 inch thick)	One row of 15 elastomeric bearing pads (Type I, 1 inch thick)

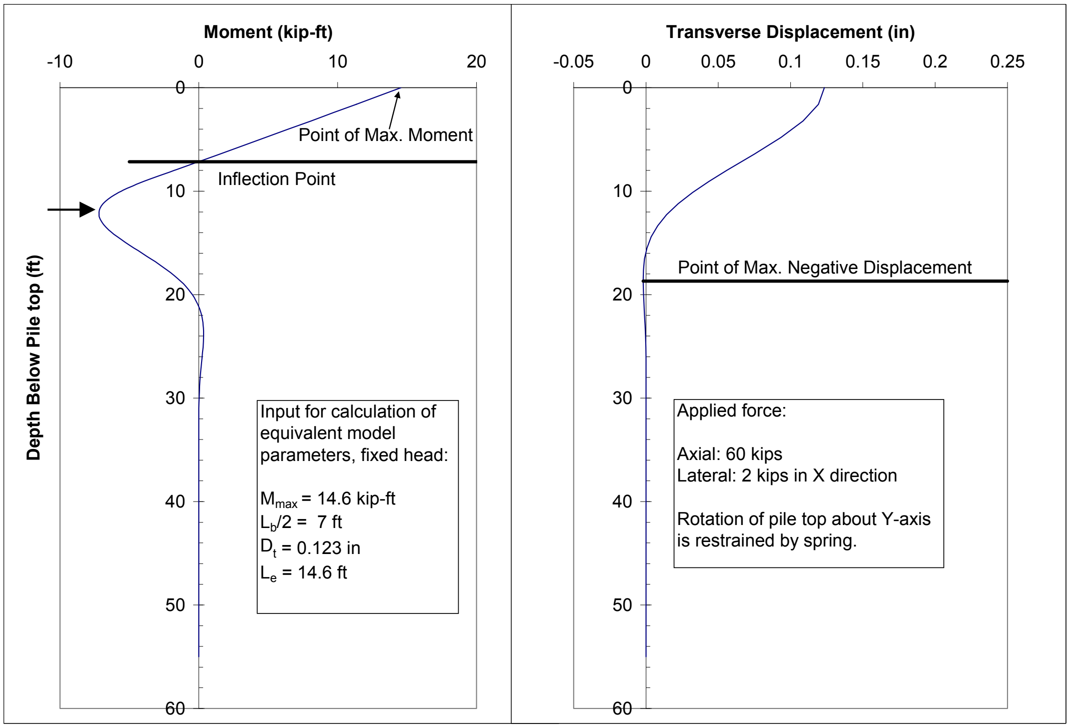

Figure 8. Results of nonlinear single pile analysis for Robeson County bridge: Moment and deflection diagram for fixed head condition.

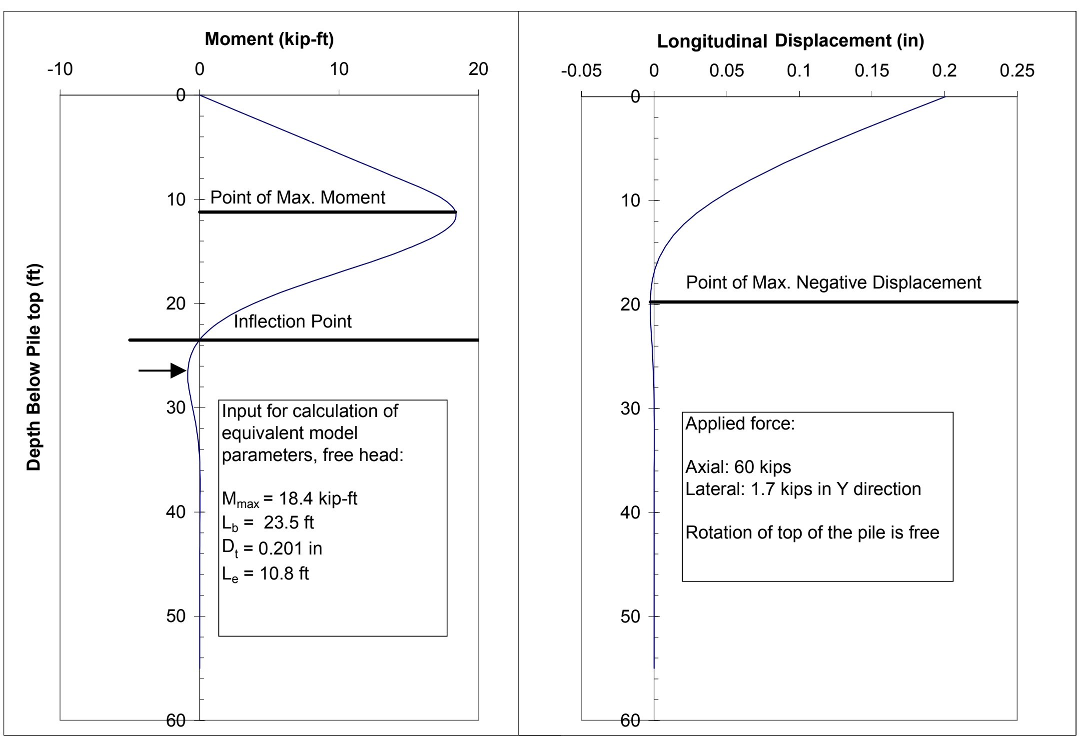

Figure 9. Results of nonlinear single pile analysis for Robeson County bridge: Moment and deflection diagram for free head condition.

Table 2. Equivalent Model Parameters for Robeson County Bridge.

Head	L_e (ft)	α	β	k
Fixed	14.5	0.95	0.26	1.0
Free	10.8	0.29	0.20	2.2

NORTHAMPTON COUNTY BRIDGE

The Northampton County bridge is a three-span bridge that crosses the CSX Railroad on US 301. Each interior bent was constructed using pipe piles with a concrete cap; the abutments are H-piles supported with wing walls. The superstructure consists of four steel girders with cast-in-place concrete slab. The interior bent of the Northampton County bridge has five 24-in. diameter steel pipe piles. Each pile has a flexural stiffness *EI* of 73,921,000 kip-in^2. The total length of the piles is 60 feet. The anticipated factored axial dead load per pile is 150 kips. The anticipated factored lateral load acting in the transverse direction is 11 kips per pile, and the anticipated factored lateral load acting in the longitudinal direction is 6 kips per pile. For this analysis, one of the two interior bents was modeled.

Lateral pile group analysis considers the spacing between the piles, which is 126 in. for this bridge, slightly greater than 5 times the 24-in. width of the pile (5D). From the FB-MultiPier manual, the leading pile's P-y multiplier is 1.0; the adjacent pile's multiplier is 0.85; and the multiplier for all other piles is 0.7. For the 5D spacing, the axial group capacity was considered to be unaffected.

Geotechnical Summary

The subsurface profile can be described as approximately 40 ft of low N-value material overlying soils whose N-value steadily increases to weathered rock at approximately 100 feet. The groundwater table was encountered approximately 8 ft below the ground surface. Because this bridge spans railroad tracks, there was no reason to consider scour effects.

A 7-ft thick layer of clayey, silty sand, which has N-values greater than 10 blows per ft, was encountered first. An approximately 21-ft thick layer of sandy clay with N-values between 0 and 5 blows per ft was reported next. From 28 to 37 ft, materials described as sandy silt were encountered. N-values of 3 to 8 blows per ft were reported for this layer. From 37 to 52 ft, clayey silty fine sand was indicated. The N-values at the top of this layer ranged from 6 to 8 blows per ft, and increased to 25 and 27 blows per ft at the bottom of the layer. Sandy clay was encountered from 52 to 62 feet. The N-values at the top of this layer were 20 blows per ft, whereas the N-values at the bottom of the layer dropped to 6 and 8 blows per foot.

The 60-foot long piles were expected to be driven either to the bottom of the clayey silty sand or to the top of the sandy clay. Materials at depths greater than 62 ft ranged from clayey silty sand with N-values between 10 and 78 blows per ft to residual silty clays with N-values ranging from 15 to 79 blows per foot. These materials grade into rock with N-values in excess of 100 blows per foot.

Point of Fixity

Results from the nonlinear single pile analysis as well as the input data for the calculation of the equivalent model parameters, described in Equations 1 through 7, are shown in Figure 10 and Figure 11 for fixed and free head conditions, respectively. The equivalent model parameters were calculated and are shown in Table 3.

Table 3. Equivalent Model Parameters for Northampton County Bridge.

Head	L_e (ft)	α	β	k
Fixed	22.2	0.94	0.37	1.2
Free	16.2	0.37	0.27	2.1

HALIFAX COUNTY BRIDGE

The Halifax County bridge spans Beech Swamp on US 301/NC 481. It is a long bridge, consisting of 9 spans with square PSC piles supporting the interior bents and H-piles supporting the exterior bents. The superstructure consists of 15 standard 3 ft by 1.75 ft PSC cored slab units.

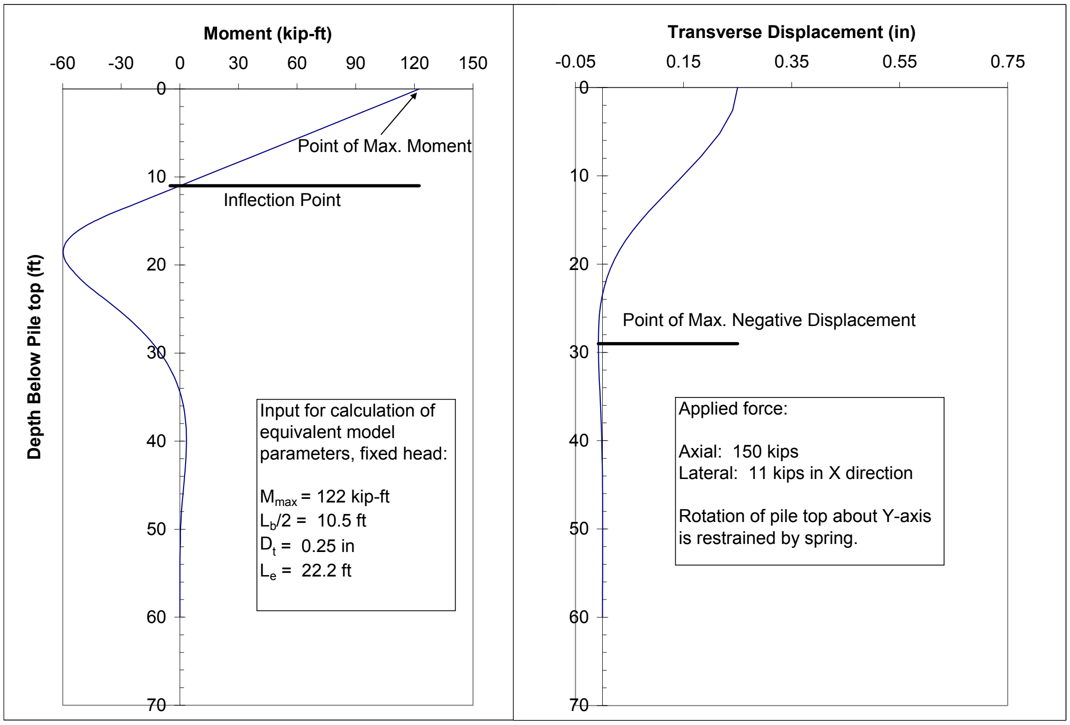

Figure 10. Results of nonlinear single pile analysis for Northampton County bridge: Moment and deflection diagram for fixed head condition.

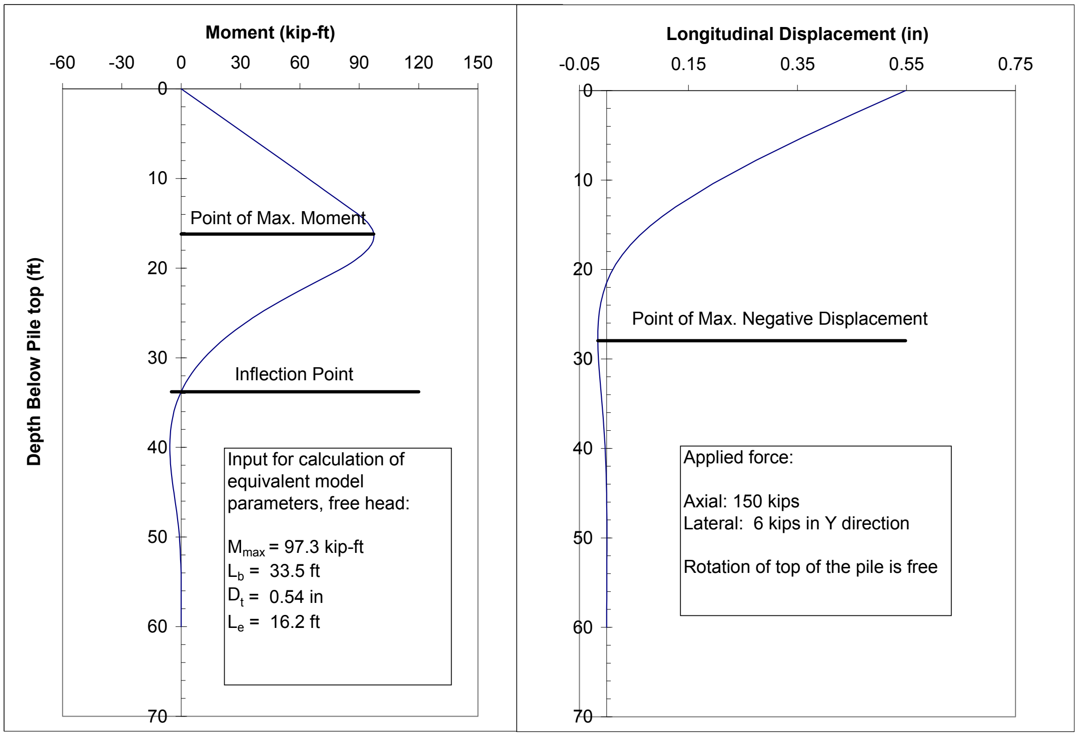

Figure 11. Results of nonlinear single pile analysis for Northampton County bridge: Moment and deflection diagram for free head condition.

The interior bent of the Halifax Bridge has eight 18-in. square PSC piles. Each pile has a flexural stiffness *EI* of 38622420 kip-in^2. The total length of the piles is 45 feet. The anticipated factored axial dead load per pile is 93 kips. The anticipated factored lateral load acting in the transverse direction is 2 kips per pile, and the anticipated factored lateral load acting in the longitudinal direction is 1.6 kips per pile. For this analysis, one out of eight interior bents was modeled.

Lateral group analysis considered the spacing between the piles, which for this bridge is 72 in., or four times the 18-in. width of the pile (4D). From the FB-MultiPier manual, the P-y multipliers were linearly interpolated between the multipliers for 3D and 5D. The leading pile's P-y multiplier is 0.9, and the adjacent piles' multipliers are 0.625 and 0.5. The multiplier for the next four piles is 0.45, and that of the trailing pile is 0.5. For the 4D spacing, the axial group capacity was considered to be unaffected.

Geotechnical Summary

The design of the interior bent foundation was based on 4 borings spaced between the 2 end bents. In general, the profile can be described as a thin layer of very loose material underlain by a more competent layer (N-value 8 to 18 blows per ft) of coarse sand. Sandy clays and silts with N-values less than 6 blows per ft extend from depths of approximately 5 to 25 feet. N-values increased to between 20 and 30 blows per ft in the Cape Fear formation, where the piles terminate.

Because the profile was relatively uniform across the site, Bent 2 was selected for the model. It should be mentioned that this bent is more representative of the majority of the interior bents, because it does not include alternating batter piles, like Bent 1 includes.

Point of Fixity

Results from the nonlinear single pile analysis, as well as the input data for the calculation of the equivalent model parameters described in Equations 1 through 7, are shown in Figure 12 and Figure 13 for fixed and free head conditions, respectively. The equivalent model parameters were calculated and are shown in Table 4.

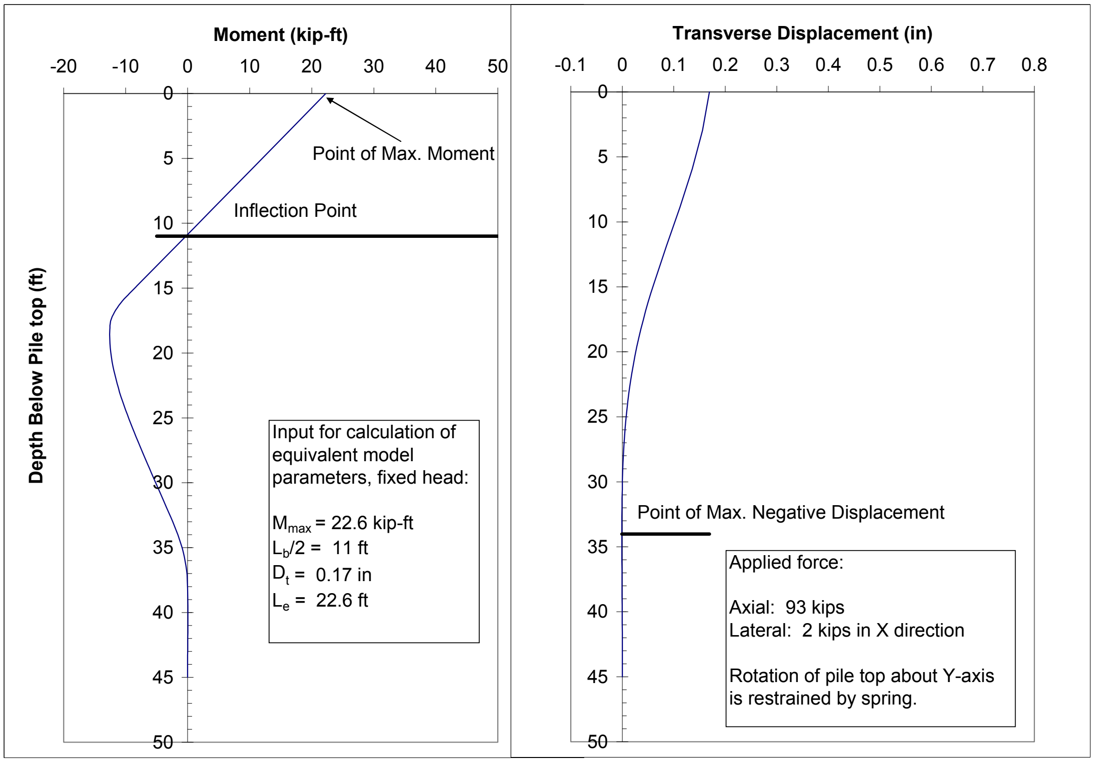

Figure 12. Results of nonlinear single pile analysis for Halifax County bridge: Moment and deflection diagram for fixed head condition.

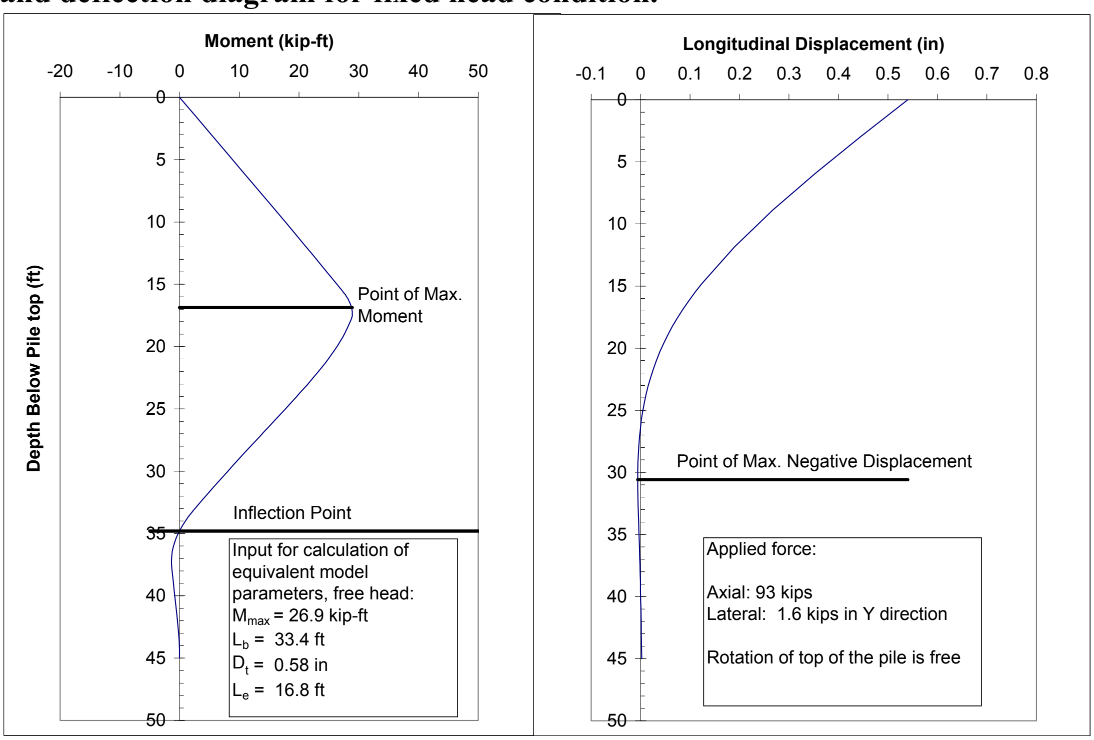

Figure 13. Results of nonlinear single pile analysis for Halifax County bridge: Moment and deflection diagram for free head condition.

Table 4. Equivalent Model Parameters for Halifax County Bridge.

Head	L_e (ft)	α	β	K
Fixed	22.6	0.98	0.57	1.0
Free	16.8	0.42	0.41	1.8

SENSITIVITY OF THE EQUIVALENT MODEL PARAMETERS

The equivalent model parameters were obtained based on the results of a nonlinear lateral single-pile analysis. In this type of analysis, both the pile and the soil respond nonlinearly under the applied lateral load. Furthermore, the presence of the axial load might magnify moments and displacements due to second order effects. Thus, it is of interest to study the sensitivity of the values of L_e, α and k to the level of axial and lateral loading applied to a nonlinear single-pile model. For the sake of brevity, only the case of the Robeson County bridge is used as illustration.

Figure 14 shows the changes in the equivalent model parameters according to the level of applied load and the corresponding top displacement for the Robeson county pile. Three levels of axial load, 0, 100, and 200% of the expected factored dead load, were applied. Under each level of axial load, the applied lateral load ranged from 0 to a maximum that caused excessive lateral displacement. Once each analysis was performed, the equivalent model parameters were computed using Equations 1 to 7. For the three bridges, the results of the sensitivity analysis show that:

i. L_e tends to increase with the increase in lateral load magnitude;

ii. α (the inertia reduction factor used to provide displacement that matches the displacement obtained from the nonlinear soil-pile model) tends to decrease with the increase in lateral load magnitude, and ranges from 1.08 to 0.4 for the fixed head condition and from 0.60 to 0.23 for the free condition;

iii. k (buckling factor) does not show a defined trend of change with the increase in lateral load magnitude, but ranges from 1 to 1.2 for the fixed head condition and from 1.6 to 2.5 for the free head condition; and

iv. The application of axial load along with the lateral load affects the response of the system by inducing second order moments that increase the lateral deflection.

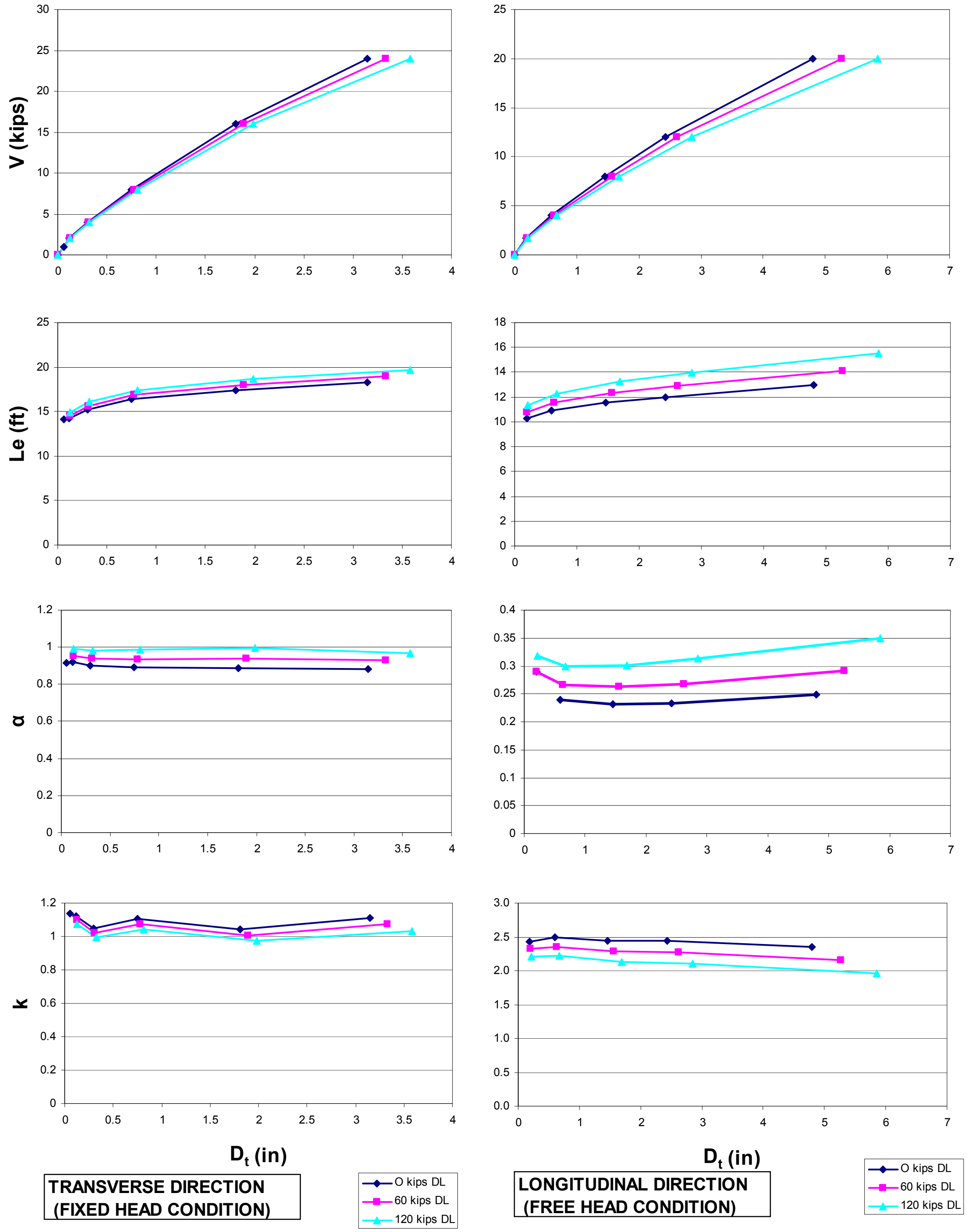

Figure 14. Sensitivity of equivalent model parameters for Robeson County bridge.

These analyses show that the level of applied loading must be carefully chosen if an equivalent model is to properly represent the nonlinear soil-pile model. If the lateral or axial load applied to the equivalent model is greater than the anticipated loading condition, then L_e and k should be larger, and α should be smaller. As such, an equivalent model that is based on equivalent parameters estimated using the higher loading condition will deflect more, will develop higher moments, and will predict less axial capacity than computerized non-linear analyses. Thus, it can be considered conservative for design purposes.

NONLINEAR VERSUS EQUIVALENT MODEL

The results of the nonlinear analyses from FB-MultiPier and SAP are compared with those from the proposed equivalent model formulations for the case study bridges. The results of the analyses using the proposed equivalent formulation are noted in Table 5 to Table 7 as *Equivalent Model*, and the results based on the point of fixity definition in practice are noted as *DOT-POF*. For each bridge, the elastic analysis of a pile bent using the proposed equivalent model involves the generation of a 3D frame model using SAP 2000. The models have the same geometry, section properties and loading as the corresponding nonlinear models. The α and β values were assigned to the pile sections as inertia and area modifier factors, respectively. In SAP, the different L_e lengths were formulated simultaneously in the transverse and longitudinal directions by creating intermediate nodes and by assigning restraints to assure fixity of the appropriate degrees of freedom. The elastic analysis with the depth to fixity values for the DOT-POF was undertaken by generating a 3D frame model in SAP. The same section properties, geometry and loading were used as in the corresponding nonlinear analysis.

Table 5 to Table 7 shows the results of the nonlinear analyses, as well as the results of the elastic analysis. For all three bridges, the proposed equivalent model properly predicts the response of the nonlinear models, as estimated using FB-MultiPier. This outcome is not surprising because the equivalent model parameters were chosen to match the results of a nonlinear lateral single pile analysis performed in FB-MultiPier using the highest expected loads in which soil, loading, and boundary conditions were modeled accordingly. The elastic analyses, based on the point of fixity definition and modeling techniques used in common practice (i.e., the DOT-POF model), overpredicted the lateral displacements in the pile bents.

Table 5. Equivalent Model Analysis Results for Robeson County Bridge

Model	Axial Disp (in)	Transverse Disp (in)	Longitudinal Disp (in)	Maximum Moment in Cap-beam (kip-ft)	Pile design forces Axial Force (kip)	Pile design forces m33 (kip-ft)
SAP (nonlinear)	0.20	0.09	0.33	172	202	5
MultiPier	0.21	0.10	0.28	182	202	0
Proposed Equivalent Model	0.22	0.10	0.22	170	203	6
DOT-POF model	0.00	0.51	0.92	105	213	9
Group	1A-LL4 Pile 3	2-WS1 Pile 1	2-WS5 Pile 6	1A-LL8	IALL4 - PILE 3	

Table 6. Equivalent Model Analysis Results for Northampton County Bridge

Model	Axial Disp (in)	Transverse Disp (in)	Longitudinal Disp (in)	Maximum Moment in Cap-beam (kip-ft)	Pile design forces Axial Force (kip)	Pile design forces m33 (kip-ft)
SAP (nonlinear)	0.23	0.26	0.89	733	341	58
MultiPier	0.25	0.23	0.72	638	353	85
Proposed Equivalent Model	0.24	0.21	0.72	743	341	77
DOT-POF model	0.00	0.64	1.76	630	375	30
Group	1A-LL5 Pile 1	2-LL1 Pile 1	2-LL5 Pile 5	1A-LL1	IA LL2 - PILE 1	

Table 7. Equivalent Model Analysis Results for Halifax County Bridge

Model	Axial Disp (in)	Transverse Disp (in)	Longitudinal Disp (in)	Maximum Moment in Cap-beam (kip-ft)	Pile design forces Axial Force (kip)	Pile design forces m33 (kip-ft)
SAP (nonlinear)	0.11	0.24	0.75	184	290	5
MultiPier	0.10	0.14	0.58	181	289	0
Proposed Equivalent Model	0.14	0.13	0.47	189	287	93
DOT-POF model	0.00	0.18	0.76	150	291	57
Group	1A-LL3 Pile 6	2-LL1 Pile 1	2-LL5 Pile 4	1A-LL8	IA LL3 -PILE 6	

The cap beam moment values predicted by the DOT-POF model are slightly lower than the values from the nonlinear analysis. This difference is due mainly to the restraint applied to the pile-cap joint nodes in the DOT-POF model, which prevents these nodes from displacing vertically. In the nonlinear models as well as in the proposed equivalent model, the piles were allowed to deform axially. For unevenly distributed live load cases, small differential vertical movements in the pile-cap joints induced rotation and, therefore, created additional moments in the cap beam and in the piles. In general, the DOT-POF models yielded significantly higher transverse and lateral displacements than the nonlinear or equivalent model. If displacement limits were in place, this would force the designer to pile sections with higher moments of inertia, and thus higher costs of pile material and installation.

SUMMARY AND CONCLUSIONS

Three bridges representing three pile types and three superstructures were selected to evaluate the current design practice that accounts for the lateral resistance of soil by introducing the point of fixity and modeling the bridge structure as an elastic frame. These modeled structures include single row H-pile, pipe pile, and PSC pile bents. The models were developed using nonlinear pile sections, elastic-plastic bent caps (using cracked moments of inertia from moment-curvature analyses), and soil along the length of each pile modeled by P-y, t-z and q-z springs.

These bridges were modeled in both SAP and FB-MultiPier programs for comparative purposes and as an independent verification tool between the two nonlinear results. Several factors regarding the applicability of the nonlinear models and the representation of the soil and structural elements have been presented. These include the toe model in FB-MultiPier and the soil stiffness parameters with corresponding magnitudes of initial shear modulus and nonlinearity of reinforced concrete elements.

Results from Multi-Pier single pile analyses were used to develop the equivalent model parameters. For each of the three bridges, sensitivity of the model to variations in axial and lateral load were explored, and the results of the two nonlinear analyses, the equivalent frame model and a point of fixity approach used in practice were compared.

Based on the results of this study, the following conclusions are advanced:

1. Overall, data from the three bridge models show good agreement between the results obtained from FB-MultiPier versus SAP for the analyzed load cases; these findings serves to verify the FB-MultiPier results and provide a better understanding of the behavior of free-standing pile bents in general.
2. Although it was not a significant issue for the three bridges investigated, differential axial deformation of the piles can play a role in moment demand in both the pile and the cap beam. If there is high differential displacement between two adjacent pile tops, high moments in the bent cap will result, leading to higher required reinforcement, as shown in the nonlinear and frame analyses where the piles are allowed to displace.
3. The results presented herein show that the equivalent frame model proposed in Robinson et al. (2006) provides results that are comparable to those obtained from both the SAP and FB-MultiPier analyses, provided that the most critical lateral load case is used to evaluate the parameters for the equivalent model. The equivalent frame model approach yields similar moments, axial loads and shear loads in the most critical case.
4. The equivalent frame model parameters are particularly sensitive to the comparable selection of both axial and lateral loads. If lateral loads used to develop the equivalent model are higher than experienced, the axial and lateral deflections, and moments will also be higher. For design purposes this is conservative.
5. The assumption of pile heads free to rotate in the bridge's longitudinal direction yields higher longitudinal displacements (and thus lower inertia reduction factors, α, in the equivalent model). If the bridge designer requires calculation of and limits to this displacement, the assumption of a fully free pile heads should be revisited. The connection between sub- and super-structure might yield enough rotational stiffness to allow some partial fixity of the pile top. This merits further research.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the efforts the North Carolina Department of Transportation (NCDOT) Structures and Geotechnical Units for funding this research, and for their technical contributions throughout the course of the work. Their assistance was valued and appreciated. However, the contents of this report reflect the views of the authors and not necessarily the views of the University, the NCDOT or the Federal Highway Administration at the time of publication. This paper does not constitute a standard, specification, or regulations. Mr. Robinson also gratefully acknowledges the USDOT's Eisenhower Fellowship program for its funding and support of his graduate studies.

NOTATION

A_p Area of the pile section.

E_p Elastic modulus of the pile material.

G_i Initial (low strain) shear modulus.

I_p Moment of inertia of the pile about the axis perpendicular to the applied load.

k Pile stability (buckling) factor

L_b Effective length for a stability (buckling) check of the pile

L_e Equivalent pile length

M_{max} Maximum moment/

P Applied axial load.

Q_b End bearing force applied to toe

Q_f Ultimate end bearing force as calculated by limit state methods.

r_o pile radius

V Lateral force applied at the top of the pile

z Axial displacement.

α Inertia reduction factor

b Area reduction factor

Δ_t Lateral displacement at the top of the pile.

Δ_z Axial displacement at the top of the pile.

ν Poisson's ratio

REFERENCES:

American Association of State Highway and Transportation Officials (AASHTO). (2007). *AASHTO LRFD Bridge Design Specifications, Customary U.S. Units*. Fourth Edition. Washington, D.C.

Borden, R.H. and Shao, L. (1995). "Construction Related Vibrations: Field Verification of Vibration Induced Settlement Model—Final Report." Report to the North Carolina Department of Transportation. 197 pages.

Bridge Software Institute (BSI). (2004). *FB-MultiPierFB-MultiPier Manual*, Version 4. University of Florida.

Chen, Y. (1997). Assessment of Pile Effective Lengths and Their Effects on Design—I. Assessment. *Computers and Structures*, Vol. 62, No. 2: 265-286.

Computers and Structures, Inc. (2003). *SAP Manual*. Berkeley, CA.

Davisson, M. T., and K. E. Robinson. (1965). Bending and Buckling of Partially Embedded Piles. *Proc. Sixth International Conference Soil Mechanics and Foundation Engineering.* Montreal: University of Toronto Press: 243-246.

Ensoft, Inc. (2004). *LPILE Manual*. Austin, TX.

Hardin, B.O. and Black, W.L. (1968), "Vibration Modulus of Normally Consolidated Clay," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 94, No. SM2, Proc. Paper 5833, March, 353-368.

Matlock, H. (1970). Correlations for Design of Laterally Loaded Piles in Soft Clay. Paper No. OTC 1204, Second Annual Offshore Technology Conference, Houston, TX. *Proceedings*, Vol. 1: 577-594.

McVay, M.C., Townsend, F.C., Bloomquist, D.G. and Caliendo, J.A. (1989). "Numerical Analysis of Vertically Loaded Pile Groups." Foundation Engineering: Current Principles and Practices. *Proceedings of the Congress. Edited by F. H. Kulhawy, American Society of Civil Engineers.* 675-690.

North Carolina Department of Transportation (2005). Geotechnical Engineering Unit, Personal Communications.

Reese, L.C., Cox, W. R. and Koop, F. D . (1974). "Analysis of Laterally Loaded Piles in Sand," Paper No. OTC 2080, Proceedings, Fifth Annual Offshore Technology Conference, Houston, Texas, (GESA Report No. D-75-9).

Reese, L.C., Cox, W. R. and Koop, F. D. (1975). "Field Testing and Analysis of Laterally Loaded Piles in Stiff Clay," Paper No. OTC 2312, Proceedings, Seventh Offshore Technology Conference, Houston, Texas.

Robertson, P.K. and Campanella, R.G. (1984). "Guideline for Use and Interpretation of the Electronic Cone Penetration Test." Hogentogler & Co., 160 pages.

Robertson, P.K., Campanella, R.G., and Wightman, A. (1983). "SPT-CPT Correlations," Journal of Geotechnical Engineering, American Society of Civil Engineers, Vol. 109, No. 11. 1449-1459.

Robinson, B., V. Suarez, P. Robalino, M. Kowalsky, and M. Gabr. (2006). Pile Bent Design Criteria. NCDOT Research Project 2005-19. https://apps.dot.state.nc.us/Projects/Research/ProjectInfo.aspx?ID=1381

Vesic, A.S. (1977). *Design of Pile Foundations*, NCHRP, Synthesis of Highway Practice #42, Transportation Research Board, National Research Council, Washington.

CHAPTER 3: Configuration Optimization of Drilled Shafts Supporting Bridge Structures: Three Case Studies

Brent Robinson, Vinicio Suarez, Mohammed Gabr, and Mervyn Kowalsky,

Previously Published as: Robinson, Brent; Suarez, Vinicio; Gabr, Mohammed; Kowalsky, Mervyn (2012) Configuration optimization of drilled shafts supporting bridge structures: Three case studies. *Practice Periodical on Structural Design and Construction*, v 17, n 3, p 93-101.

ABSTRACT: A common approach to estimate the point of fixity is to utilize the results of a single pile lateral analysis. Although no universal agreement exists as to the definition of the location of the point of fixity, it is generally accepted that its location will impact the computed stresses and displacements of a bridge structure. Work in this study summarizes a method to determine the cantilever's equivalent length of drilled shaft foundation elements supporting a bridge. Results from an equivalent frame model are compared to those for bents modeled using the finite element method and nonlinear soil models for three bridges in North Carolina. Results indicated that the equivalent frame model provides responses that are comparable to those obtained from more rigorous finite element analyses. The study presents the results of the optimization of the support system by reducing the number, or size, of the shafts while maintaining acceptable level of safety.

CE Database subject headings: Bents, Bridge design, Bridge foundations, Case reports, Drilled shafts

INTRODUCTION

In practice, a bridge bent structure often can be modeled as a linear frame that takes into account the effect of supporting foundation by extending the frame's columns to a presumptive depth below the ground surface where deformation of the foundation elements is negligible. The bottom point of such presumptive depth is often referred to as the *point of fixity*. Geotechnical engineers analyze the foundation elements and suggest pile or shaft sizes and lengths for given sets of axial and lateral loads. Structural engineers determine the required loads according to code (e.g. AASHTO dead, live, wind, impact loading, etc.) and design the bent caps as well as the remainder of the bridge structure considering a free-standing elastic frame subjected to various AASHTO load combinations. Such an approach is usually adopted because many available frame analysis/design programs (but especially the design-type software) do not allow for input of springs that represent soils and foundation; thus, a single point of fixity for each foundation element is used to allow the bridge to be represented in a simple frame configuration.

Point-of-fixity equations were proposed by Davisson and Robinson (1965) and are now commonly used due to their inclusion in the AASHTO LRFD Bridge Design Specifications (2004). However, the use of these equations in the AASHTO specifications is only recommended for the assessment of the buckling effective length of the columns that constitute the frame members. Generally, such point of fixity models do not distinguish between free- and fix-headed piles and, therefore, do not yield accurate lateral displacements for the pile bent (albeit, this was not their intended application).

POINT OF FIXITY APPROACH

A common approach to estimate the point of fixity is to utilize the results of a single shaft/pile lateral analysis (NCDOT 2005) and assign a point of fixity based on the deformed shape. Some engineers have used the point of maximum negative moment or the point of maximum negative displacement along the shaft/pile length to define a point of fixity and perform elastic frame analysis of the bridge structure system. Although no universal agreement exists as to the definition of the location of the point of fixity, it is known that its location will impact the computed stresses and displacements of a bridge structure. For example, given a foundation element of a fixed diameter and applied lateral load, a deeper point of fixity leads to higher bending stresses and larger displacements. To withstand these stresses and reduce displacements, a more conservative design is specified due to increasing the size of the foundation elements.

Robinson et al. (2006) observe that analysis of a cantilevered column with an *equivalent* length determined by the point of fixity does not yield results that match the magnitudes of maximum moments, lateral pile top displacements, or buckling behavior obtained from nonlinear single pile lateral analyses. To model a combined head condition, an equivalent column model was proposed by Robinson et al. (2007) as follows:

<u>Free head condition (longitudinal direction)</u>

$$L_{e,free} = \frac{M_{\max,free}}{V_{free}} \tag{1}$$

$$\alpha_{free} = \frac{L^3_{e,free} V_{free}}{3E_p I_p \Delta_{t,free}} \tag{2}$$

<u>Fixed head condition</u>

$$L_{e,fix} = \frac{2M_{\max,fix}}{V_{fix}} \tag{3}$$

$$\alpha_{fix} = \frac{L^3_{e,fix} V_{fix}}{12E_p I_p \Delta_{t,fix}} \tag{4}$$

where:

L_e The length of a cantilever fixed at the base that will develop the same maximum moment, M_{max}, as the nonlinear soil-pile model under the lateral load, V.

M_{max} Maximum moment from the single pile lateral analysis for a head condition.

V Maximum lateral force applied at the top of the pile for a particular head condition.

α Inertia reduction factor that when multiplied by the pile inertia, I_p, in the equivalent model, will result in the same lateral displacement as the nonlinear model.

E_p Elastic modulus of the pile material.

I_p Moment of inertia of the pile about the axis perpendicular to the applied load.

Δ_t Displacement at the top of the pile from the single pile lateral analysis caused by the application of the lateral load.

Robinson et al. (2007) proposed equations 1 through 4 to calculate a point of fixity using the moments and displacements estimated from a single shaft/pile lateral analysis, such as those available in LPILE (Ensoft, 2008) or MultiPier (Bridge Software Institute, BSI, 2004). To capture the disparate behavior in the transverse and lateral directions, single shaft/pile analysis can be run twice, once with a free head condition (longitudinal) and once with a fixed head condition (transverse). This approach, however, leads to two points of fixity; the deeper of the two may be used for a conservative analysis.

In this paper, a series of three bridge structures with drilled shaft foundations is analyzed using the Robinson et al (2007) point-of-fixity approach, and the results are compared to a more detailed 3-D numerical analysis. Modeling of the bridge structures is performed within the framework of the MultiPier and SAP 2000 suite of numerical analyses programs. Results from both programs are compared with those obtained from the simplified point-of-fixity approach. Issues related to conservatism and to the impact of design assumptions are discussed.

MODELING SOFTWARE

MultiPier is selected as the primary modeling tool for analysis because it: (1) has an interactive bridge bent software wizard built in; (2) automatically models the soil resistance (lateral and axial, single and group) using methods that represent the current state of practice; (3) allows for typical linear or nonlinear bent cap models to be utilized; and (4) has the option of modeling bridges with multiple bents connected by idealized superstructure elements. In addition, the finite element program SAP 2000, Version 8.2.5 (Computers and Structures Inc., 2003) is used to analyze the same case studies as an independent check of the results. Using SAP 2000, models for each bent of the three case studies are created, both as point of fixity-type frame analysis, and with full-length shafts with nonlinear springs along the length to model shaft-soil responses. The practical limitations of each numerical program are discussed by Robinson et al. (2006).

AASHTO LOAD CASES

The allowed number of load cases in MultiPier software includes one dead load case, up to five wind cases, and nine live load cases, longitudinal forces, impact forces, or other cases. MultiPier also includes a built-in wind load generator. SAP has no such limitations on the number of load cases, but the load cases are not built in the software. In this paper, AASHTO loading groups I, II and III, which consider dead and live loading conditions, wind loading conditions and live load and braking forces, respectively, are used in the analysis. Additional AASHTO load cases are not considered, as they are not applicable for the configuration of the three bridges considered herein. The point-of-fixity model and MultiPier results were synthesized for comparison to the SAP results using the following parameters:

a. Maximum foundation top lateral displacements in the transverse, longitudinal, and axial directions, the maximum moment in the bent cap, and the maximum axial force, moment and demand/capacity ratio in the foundation elements;
b. For each maxima, identification of the AASHTO load case and (if applicable) drilled shaft number associated with the maxima; and
c. Use of SAP to check the same load case from MultiPier for the desired maxima and displacement, moment or force envelope.

For both the MultiPier and SAP analyses, the concrete bent cap models were entered as linear elastic. Because the amount of reinforcement for the "already" designed bridges was known, a cracked moment of inertia was estimated. As this cracked moment of inertia was entered into both SAP and MultiPier, the maximum moments could be identified and the required amount of rebar checked.

The axial forces and moments are used in MultiPier, along with the nonlinear pile section properties, to calculate a demand/capacity ratio (DCR). As discussed in the MultiPier manual (BSI 2004), the DCR "is an estimate of the percentage of the cross sections' capacity that has been reached for that particular loading state [due a particular load case]." A DCR of 1.0 or greater implies failure. A DCR of less than 1 implies the section is able to sustain the range of AASHTO factored load cases entered for the analysis.

Table 5. List and Description of Cases Studies: Locations and Construction Details

Bridge Name and Location	Bents and Foundation		Bent Cap Dimensions and Reinforcement		Bent Super/Substructure Connection	
	Interior Bents and Foundation Type	End Bents and Foundation Type	Interior Bents	End Bents	Interior Bents	End Bents
Rowan County Bridge. One spans: (170 feet) with 2 end bents	None	Four 48-inch diameter (nominal) drilled shafts reinforced with 20 #11 bars and spaced 24 feet apart. One HP12x53 brace pile is placed for the wing wall.	None	83.5 ft long by 54 inch wide by 60 inch deep (minimum) Class A concrete beam with wing walls	None	Five elastomeric bearing pads, 3-13/16 in. thick, Type VI (one end bent fixed, one end bent expansion)
Wake County Bridge, four spans (100.8, 98.4, 98.4 and 100.8 feet)	3 bents, seven 4.5 ft (1372 mm) diameter drilled shafts, spaced 22.3 ft (6.8 m), free length cast with 4 ft (1220 mm) diameter, 39.4 feet (12 m) long columns	28 HP12x53 (310x79); Eight brace piles battered 1:4	56 inch (1420 mm) wide by 51 inch (1300 mm) deep by 144 ft (43.8 m) long Class A concrete beam. Seven #36 (metric) bars (top and bottom), four #16 (metric) on each face	160 ft (48.7 m) long by 49 inch (1250 mm) wide by 30 inch (760 mm) deep (minimum) Class A concrete beam with wing walls	Two rows of 17 elastomeric bearing pads (Type V pads with 2 inch diameter anchor bolts)	One row of 17 elastomeric bearing pads (Type V, 2-1/4 inch or 57 mm thick)

Table 5 Continued.

Bridge Name and Location	Bents and Foundation		Bent Cap Dimensions and Reinforcement		Bent Super/Substructure Connection	
	Interior Bents and Foundation Type	End Bents and Foundation Type	Interior Bents	End Bents	Interior Bents	End Bents
Pitt County Bridge, 20 Spans, (Span between end bent 1 and Bent 1 is 101 ft-1 inch, 14 are 100 feet, four are 90 feet, final span is 91 ft-1 inch)	Two five ft (1524 mm) diameter drilled shafts, reducing to 4.5 ft (1.37 m) diameter for the free column above the ground surface, spaced 19.5 ft (6 m). Cast after shafts are completed with 4.5 ft (1.37 m) diameter, 35 feet (10.67 m) long free standing columns	Two Bents with H piles	62 inch (1575 mm) wide by 48 inch (1219 mm) deep by 30 ft (9.14 m) Class A concrete beam. Cap Reinforced with: Eight #11 bars on top, eight #11 bars on the bottom, and eight #5 on each face.	39.25 ft (12 m) long Class A concrete beam with wing walls	Two rows of four elastomeric bearing pads (Type V pads with 2 inch diameter anchor bolts)	One row of four elastomeric bearing pads (Type V, 2-1/4 inch or 57 mm thick)

CASE STUDIES

Three case studies with drilled shafts as the bridge foundation were selected for this research. Table 5 summarizes the location, type of superstructure, type of foundation, and dimensions and reinforcement details of the cap beams. In these analyses, the transverse direction is parallel to the bridge's cap beam, and the longitudinal direction is perpendicular to the bridge's cap beam. The axial direction is perpendicular to the cap beam and parallel to the axis of the vertical deep foundation.

ROWAN COUNTY BRIDGE

The Rowan County Bridge is on US 70 in North Carolina and passes over a Norfolk Southern Railroad line. This bridge consists of a single span with two end bents supported by drilled shafts. Drilled shafts were chosen over driven piles in part due to the magnitude of loads imposed by the span length, but also to minimize vibration and disturbance to the active railroad track. The superstructure consists of five steel girders fabricated from AASHTO M270 Grade 50W material. Girder flanges are 20 in. wide x 2 in. thick, and the girder web is 77 in. high x 5/8 in. thick.

Geotechnical Summary

For this case study, the western-most end bent was modeled. The soil profile at this bent shows the ground water level to be approximately 18 ft below the ground surface. Up to 9 ft below the ground surface, medium stiff silty sandy clay was reported. The Standard Penetration Test (SPT) N-value for this layer is 5 blows per foot. Next, medium stiff to stiff micaceous clayey sandy silt was encountered from 9 to 20 ft, with N values averaging 8 blows per foot. Medium to very dense silty sand was reported from 20 to 31 ft, with N values of 19 and 58 blows per foot. Weathered rock with N values in excess of 100 blows per foot was observed between 31 and 40.5 ft, where the boring terminated.

As designed, the shafts were to terminate in the weathered rock. Based on the project data, capacities along the shaft and at the toe were estimated for the preliminary analysis using drilled shaft t-z and Q-z models, originally proposed by O'Neill et al. (1996) for the weathered rock and by BSI (2006) for drilled shafts in sands. P-y models for lateral shaft resistance in sand were developed according to Reese et al. (1974), and the weathered rock model utilizes the limestone model by McVay (2004). Lateral group analysis considers the spacing between the shafts, which for this bridge is 24 ft, or 6 times the 4 ft diameter of each shaft (6D). Thus, the P-y multipliers were set to 1. For the 6D spacing, axial group capacity was considered to be unaffected.

Equivalent Model

After a single shaft lateral analysis was performed in MultiPier, the equivalent model parameters were calculated based on the proposed procedure by Robinson et al. (2006), which is summarized in Equations 1 through 4. The equivalent model parameters, including effective length for lateral loads and displacements, are shown in Table 6. These effective lengths were then input into a SAP frame, without soil, and are presented in the next section as "SAP—Equivalent" results.

Table 6. Equivalent Model Parameters for Rowan County Bridge

Head Condition	Le (ft)	α
Fixed	36.3	0.92
Free	22.9	0.23

Analysis Results—SAP and MultiPier

Figure 15 shows the MultiPier model and the discretization of the drilled shaft and the soil profile. Figure 16 shows a comparison of lateral deformation and moment deformation with depth estimated using three different approaches. The first is from MultiPier model, the second is from linear frame analysis using SAP but with input using the POF parameters as is defined in Table 2 (SAP with Equivalent POF), and the third approach is using a definition of POF that is based on locating it at the point of maximum negative moment. Figure 16 shows a good agreement between SAP with Equivalent POF (with the Le and alpha given in Table 6) and results from MultiPier. While such good agreement is not surprising since the equivalent model parameters were obtained from MultiPier single shaft analysis, the results point out that simplified model cannot capture the non linear soil response. In Rowan County Bridge, the bents have 5 and 7 piles respectively. Since the bearings are not directly on top of the shafts, the dead load produces moments in the shafts in addition to the moments generated by the lateral loads. Although this loading condition and group effects (P-y multipliers) were not accounted in the single pile analysis that yielded Le and α, it seems that the proposed equivalent model produces reasonable results for the design of the pile sections. Results also show that the use of POF that is based on locating it at the point of maximum negative moment yielded non conservative response in the case of Rowan County bridge parameters.

Results for the lateral response are summarized in Table 7. This table compares the predicted maximum moment, shear and axial loads in a particular drilled shaft foundation. The ratio of the demand placed on the shaft due to the AASHTO load cases to the capacity of the shaft based on the combined axial force and moment capacity is also shown.

Table 7. Rowan County Nonlinear Analysis Results: Maximum Lateral Load Case

Model	Maximum Moment (kip-ft)	Maximum Axial Force (kips)	Maximum Shear Force (kips)
SAP—Nonlinear	605	257	28.7
MultiPier	529	249	28.7
SAP—Equivalent	573	253	27.7
Controlling AASHTO Group	AASHTO Group II Shaft 4, Demand Capacity Ratio = 0.171		

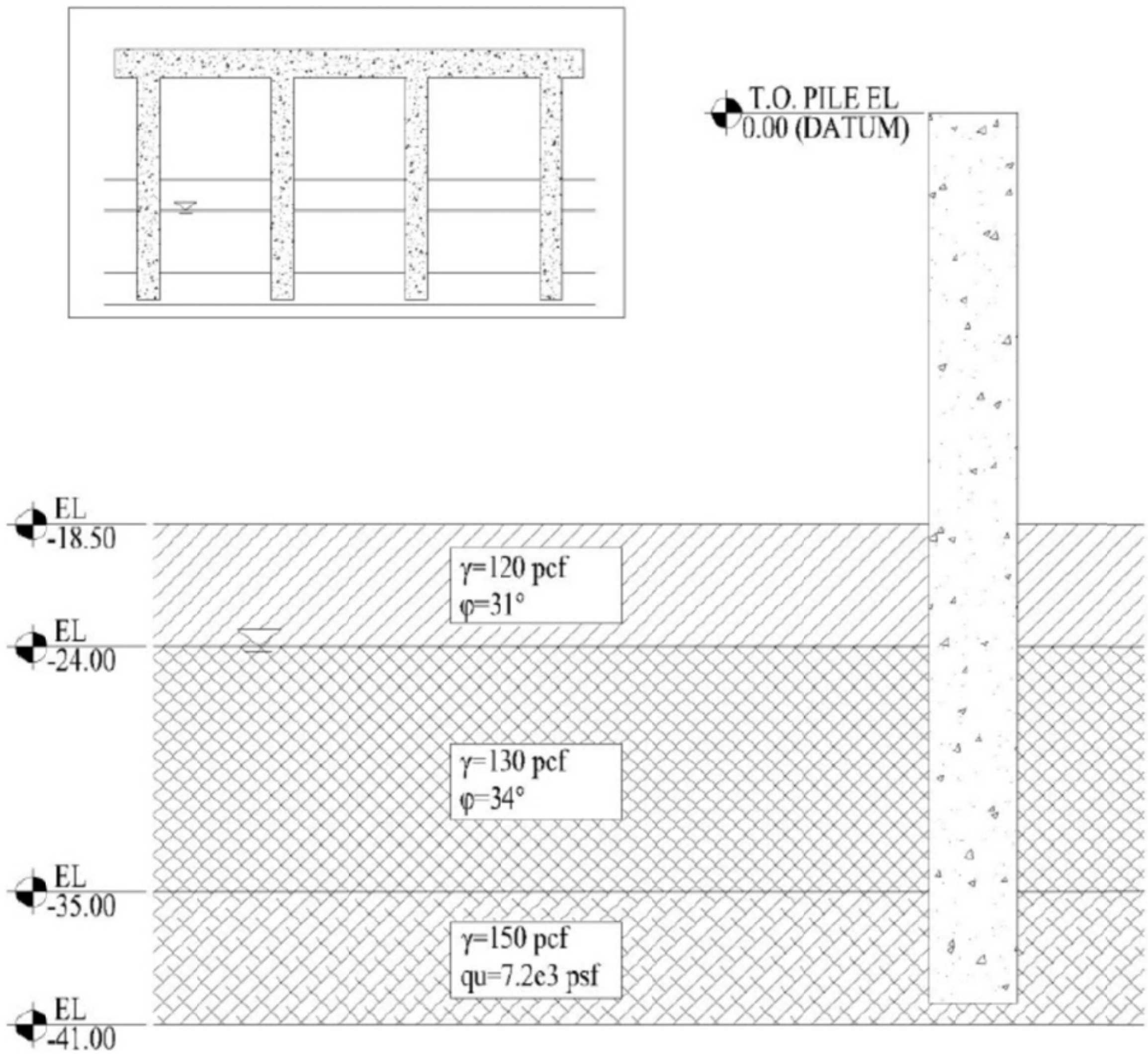

Figure 15. MultiPier model--Rowan County bridge model for single drilled shaft and the bridge bent (inset).

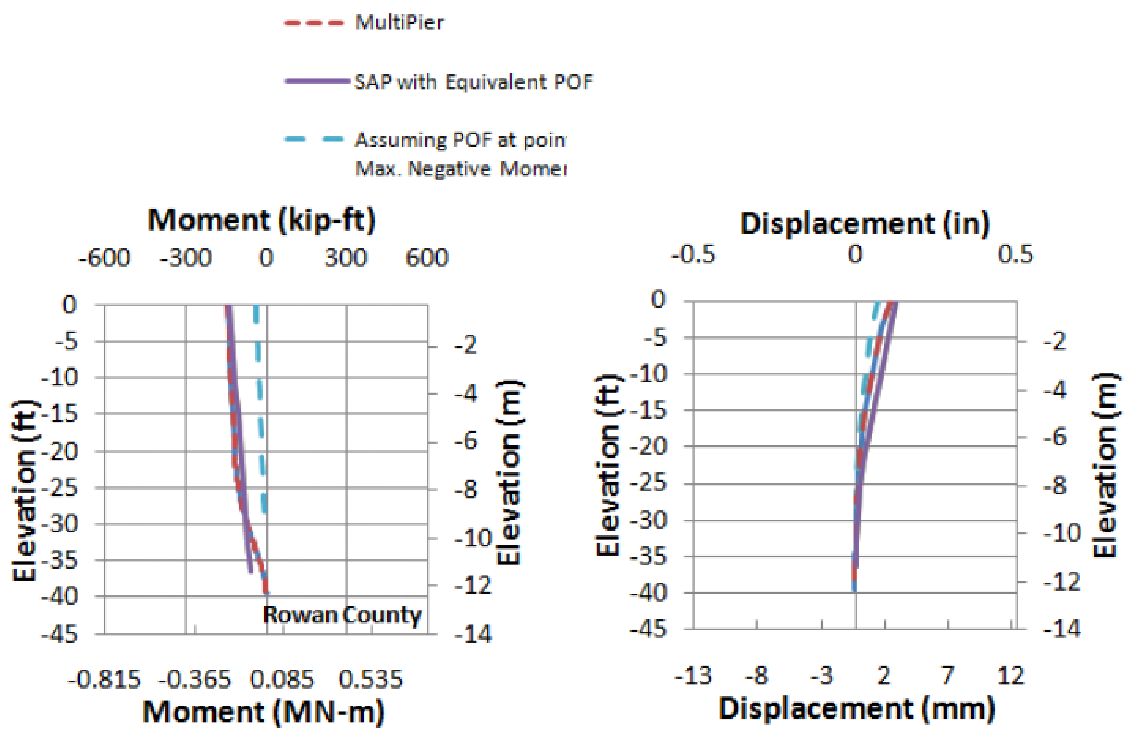

Figure 16. Distribution of Bending Moment and Lateral Deformation with Depth for a Single Pile-Rowan County

Data in Table 7 show reasonable agreement between the two models (SAP—Nonlinear and MultiPier) which fully account for the presence of soil around the shafts. Differences between the two nonlinear models could be due to the way the P-y and t-z models are generated in MultiPier (at each discrete node) versus SAP, which is linearly interpolated with the depth between the P-y or t-z curve at the top of a layer, and the P-y or t-z curve at the bottom of a layer. Also, the SAP model, which uses the equivalent point-of- fixity method as described by Robinson et al. (2006), produces reasonable results compared to the MultiPier results.

Optimization of Design

Further MultiPier analysis was performed to optimize the design by reducing the size or number of the shafts in the bridge bent. The bent was constructed using four 48 in. diameter drilled shafts with a reinforcement ratio of 1.6%. First, the four 48 in. shafts were replaced by

42, 36 and 30 in. shafts with reinforcement ratios of 2%. Finally, the number of 48 in. shafts was reduced from four to three, with the 1.6% reinforcement ratio remaining constant.

A summary of the optimization results is shown in Table 8. The values shown are the maxima over all the AASHTO load cases analyzed (Groups I, IA, II, and III). As expected, the ratio of the demand placed on the shaft to its capacity, based on axial loads and moments applied, steadily increases as the shaft diameter decreases. Similarly, transverse, longitudinal and vertical displacements steadily increase as the shaft diameter decreases. It seems, however, that the number of shafts could be reduced to three while still maintaining an acceptable margin of safety in terms of capacity demand and deformation levels. The reduction in diameter to 36 in. also seems to be viable, but most likely such flexibility will be controlled by construction and specifications issues.

Table 8. Rowan County Alternative Shaft Configurations

	48 in. shaft, 1.6% reinf.	42 in. shaft, 2% reinf.	36 in. shaft, 2% reinf.	30 in. shaft, 2% reinf.	Three 48 in. shafts, 1.6% reinf.
Demand/Capacity Ratio (Shafts)	0.192	0.352	0.423	0.539	0.397
Displacement, transverse (Shaft top, in.)	0.13	0.16	0.24	0.41	0.24
Displacement, longitudinal (Shaft top, in.)	0.26	0.36	0.58	1.17	0.38
Displacement, axial (Shaft top, in.)	0.27	0.34	0.46	0.67	0.49

WAKE COUNTY BRIDGE

This Wake County, North Carolina case study analyzes a bridge that spans Richland Creek on the NC 98 bypass between US 1 and US 1A. This concrete girder bridge is supported by three interior drilled shaft bents and two end bents with vertical and battered HP 12x53(HP 310x79) driven piles. The superstructure consists of seventeen 4.5 ft (1372 mm) prestressed concrete girders with a cast-in-place concrete deck slab**.** Diaphragms are constructed between the girders at the end bents, the interior bents and between bents.

Geotechnical Summary

One of the three interior drilled shaft bents was modeled for this study. The soil profile can be characterized generally as residual and composed of approximately 10 ft (3.2 m) of low N-value material overlying weathered and parent rock. The groundwater table was encountered approximately 10 in. (0.25 m) below the ground surface. Scour effects are considered in this model because the bridge crosses a small stream that has a relatively high flood elevation.

The soil borings indicate a 10 ft thick layer of clayey silt and sandy clay that has N-values between 7 and 17 blows per foot. Below that, weathered or severely fractured black gneiss was encountered. Rock recovery ratios were between 0 and 45% in the weathered material, and generally 100% in the parent material. Rock Quality Designation (RQD) values were typically 0% in the weathered rock and between 50 and 90% in the sound gneiss.

According to the project plans, the 30 to 45 ft (9 to 14 m) long shafts were installed with a socket in the weathered and parent rock material. After the shafts were completed, a 35 ft (11 m) column was cast on top of each shaft with a reduced diameter. In MultiPier, the profile soils were modeled using the Reese and Welch (1975) P-y curves for stiff clay below the water table, with a 4 ksf unconfined compressive strength for the weathered rock, and using the McVay (2004) limestone model for P-y curves of the parent material. Axial t-z curves were developed using the O'Neill and Reese (1999) model for drilled shafts in intermediate geomaterials.

Equivalent Model

The equivalent model parameters are shown in Table 9. These parameters were then input into a SAP frame without soil and are presented in the next section as "SAP—Equivalent."

Table 9. Equivalent Model Parameters for Wake County Bridge

Head	Le (ft)	α
Fixed	67	0.85
Free	29.42	1.16

Analysis Results—SAP and MultiPier

A model set-up of the Wake County case study is shown in Figure 17. Similar to the results shown for Rowan County, Figure 18 shows a good agreement between SAP with Equivalent POF (with the Le and alpha given in Table 9) and results from MultiPier. The POF approach based on the point of maximum negative moment produces also good results in terms of moment but consistently underpredicts lateral deformation. Table 10 shows the maximum moment, shear and axial forces developed in the most critical shaft for each model under the maximum lateral load case. Both the MultiPier and SAP models yield consistent shaft responses under the applied load cases.

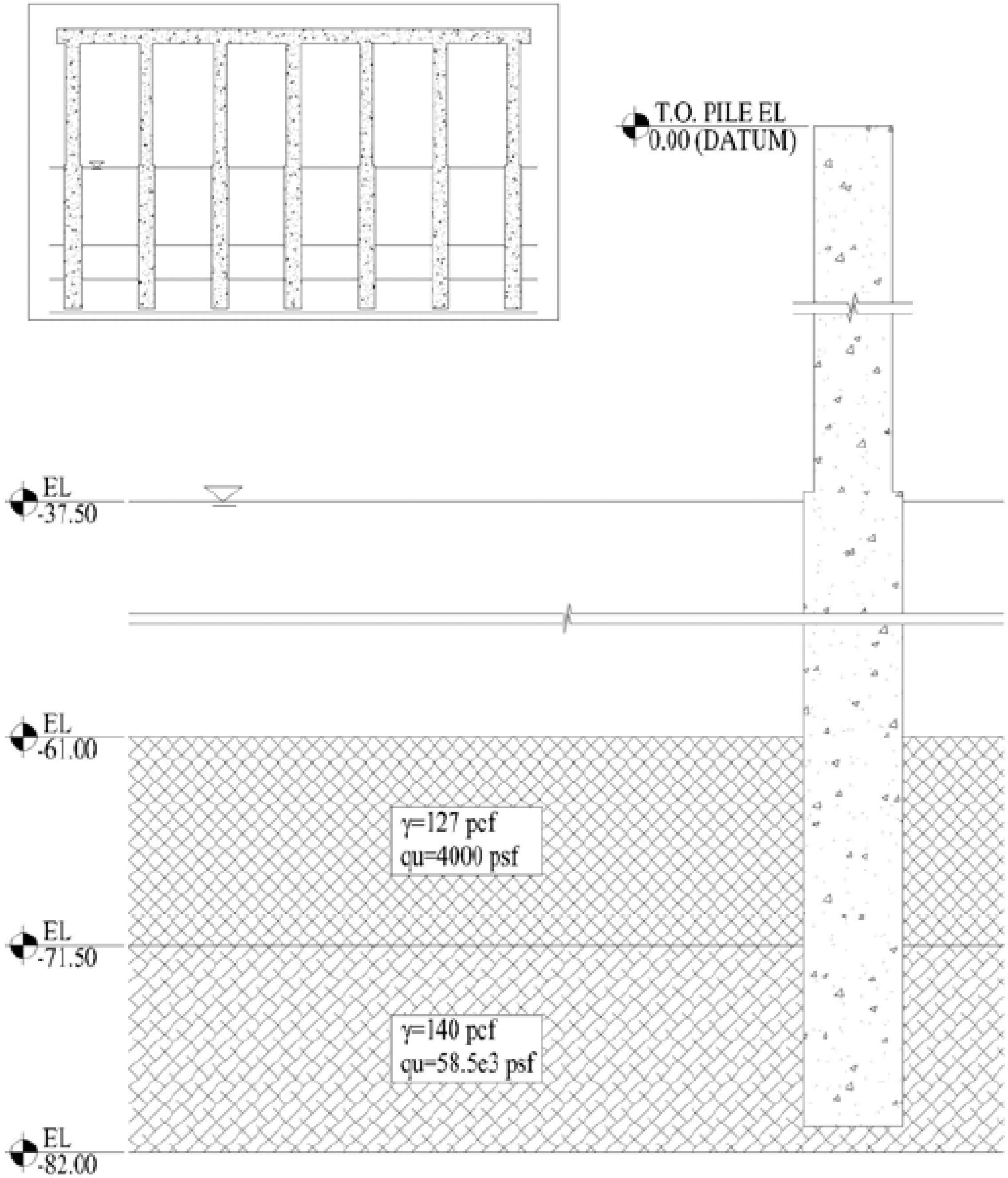

Figure 17. Wake County bridge MultiPier model for a single drilled shaft and the bridge bent (inset).

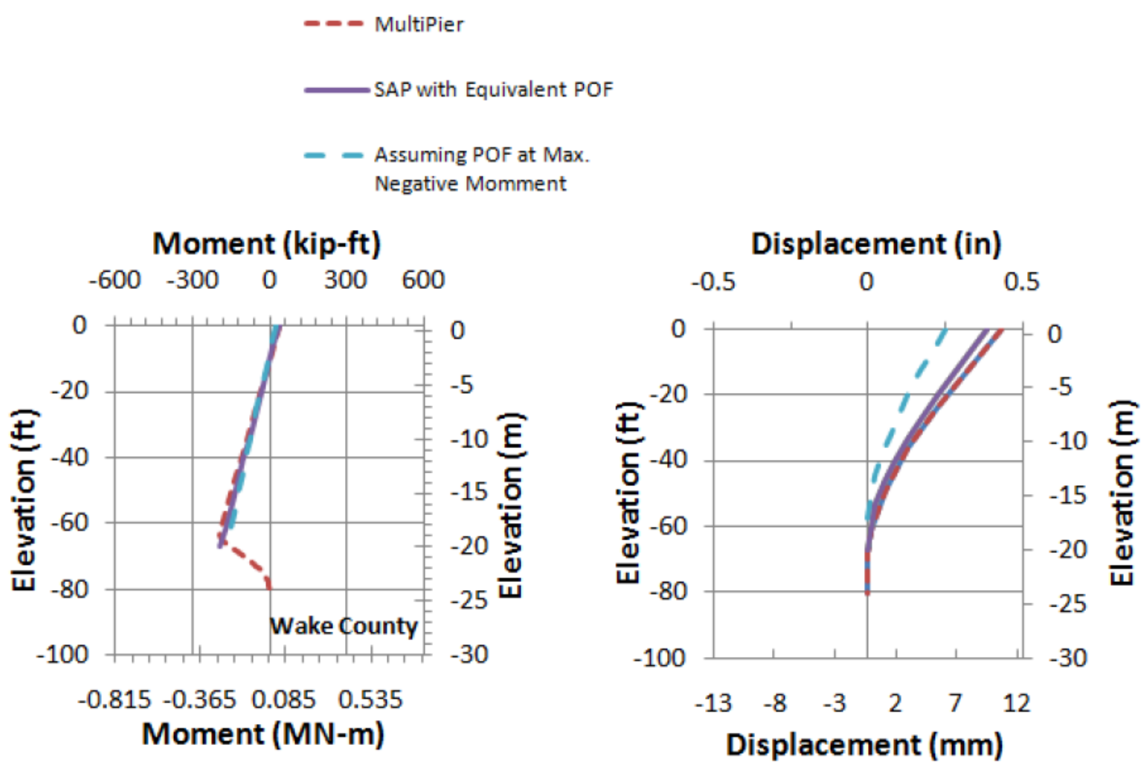

Figure 18. Distribution of Bending Moment and Lateral Deformation with Depth for a Single Pile-Wake County

Table 10. Wake County Nonlinear Analysis Results at Top of Shaft: Maximum Lateral Load Case

Model	Maximum Moment (kip-ft)	Maximum Axial Force (kips)	Maximum Shear Force (kips)
SAP--Nonlinear	442	4052	40
MultiPier	450	4100	45
SAP—Equivalent	505	4053	41
Controlling AASHTO Group	AASHTO Group II Shaft 4, Demand Capacity Ratio = 0.226		

Optimization of Design

The analysis was run with a reduction of the number or size of the shafts while maintaining the same load cases. For this bridge, seven 54 in. diameter drilled shafts were used. Table 11 shows the results obtained from reducing the number of shafts. DCRs tend to be between 0.2 and 0.3, although displacements in the transverse and longitudinal directions begin to exceed 1 in. when 42 in. diameter shafts are considered. The highest DCR in the original layout occurs due to the reduction in the shaft area between the shaft and the column. In this case, it seems that seven 48 in. shafts provide an adequate safety margin for capacity and deformation.

Table 11. Wake County Alternative Shaft Configurations, based on AASHTO Groups I, IA, II and III

	Seven 54 in. shafts, 48 in. columns	Seven 48 in. shafts uniform	Seven 42 in. shafts uniform	Seven 36 in. shafts uniform	Seven 30 in. shafts, uniform	Six 42 in. shafts uniform
Demand/Capacity Ratio (Shafts)	0.32	0.21	0.26	0.36	0.49	0.29
Displacement, transverse (Shaft top, in.)	0.43	0.47	0.75	1.28	2.5	0.91
Displacement, longitudinal (Shaft top, in.)	0.67	0.75	1.22	2.18	4.4	1.46
Displacement, axial (Shaft top, in.)	0.24	0.24	0.28	0.38	0.53	0.35

PITT COUNTY BRIDGE

The Pitt County Bridge spans the Tar River and its overflow area on North Carolina SR 1565. It consists of 2 end bents and 19 interior bents, for a total length of 1952 feet. The superstructure consists of four 54 in. prestressed concrete girders with a cast-in-place concrete deck slab. Diaphragms are constructed between the girders at the end bents, the interior bents and between bents.

Geotechnical Summary

For this analysis, one of the three interior drilled shaft bents was modeled. The profile is typical of the coastal region, consisting of alternating layers of sands and clayey silts. At a depth of approximately 25 ft, the Pee Dee formation was encountered. Generally, this stratum consists of soils described as sands or sandy silts with intervals of sandy limestone.

The boring indicates a 3 ft thick layer of clayey silt that has an N-value of 2 blows per foot. This is underlain by saturated fine sands and wet sandy clayey silts that extend to a depth of 24 ft, with N-values between 8 and 15 blows per foot. The Pee Dee formation follows the saturated fine sand; it is described as fine to coarse sand, fine to coarse sandy silt, fine sandy clay and fine sandy clayey silt, all with interbedded layers of sandy limestone. The limestone layers tend to be no more than 12 in. thick, and are indicated in the boring log. N-values are highly variable depending on whether the SPT struck a limestone layer (N-values greater than 100 blows per foot) or one of the sandy layers (N-values of around 10 to 20 blows per foot). In this boring, the Pee Dee formation reportedly extends from a depth of 24 ft to the termination of the boring at 98.9 feet.

The shafts at Bent 4 were expected to be installed to depths of 100 ft (30.5 m) below the ground surface, well into the Pee Dee formation. After the shafts were completed, a 37 ft (12 m) column that has a reduced diameter was cast on top of the shaft. In MultiPier, the upper 25 ft of the soil profile was removed for scour considerations. The soils were modeled using the Reese et al. (1974) P-y curves for sand, with a 32^{o} friction angle in the Pee Dee formation's soils and 45^{o} friction angles in the sandy limestone. Axial t-z curves were developed using the BSI (2004) model for drilled shafts in sands.

Equivalent Model

The equivalent model parameters are shown in Table 12. These parameters were then input into a SAP frame without soil and are presented in the next section as "SAP—Equivalent."

Table 12. Equivalent Model Parameters for Pitt County Bridge

Head	Le (ft)	α
Fixed	73.4	0.30
Free	84.2	0.47

Analysis Results—SAP and MultiPier

The MultiPier model is shown in Figure 19. The distribution of bending moment and lateral deformation with depth for a single shaft is shown in Figure 20. Data show no significant difference in maximum moment magnitude, however the lateral displacement using the assumption of POF approach based on the point of maximum negative moment is less as compared to the results from MultiPier. This response was observed for the three case studies mainly because no inertia reduction (α) was used to match the stiffness of the MultiPier (nonlinear) model using the approach of POF based on the point of maximum negative moment. Table 13 shows the maximum moment, shear and axial forces developed in the most critical shaft for each model. Similar to the two previous case studies, both the MultiPier and SAP models yield consistent responses under the applied load cases.

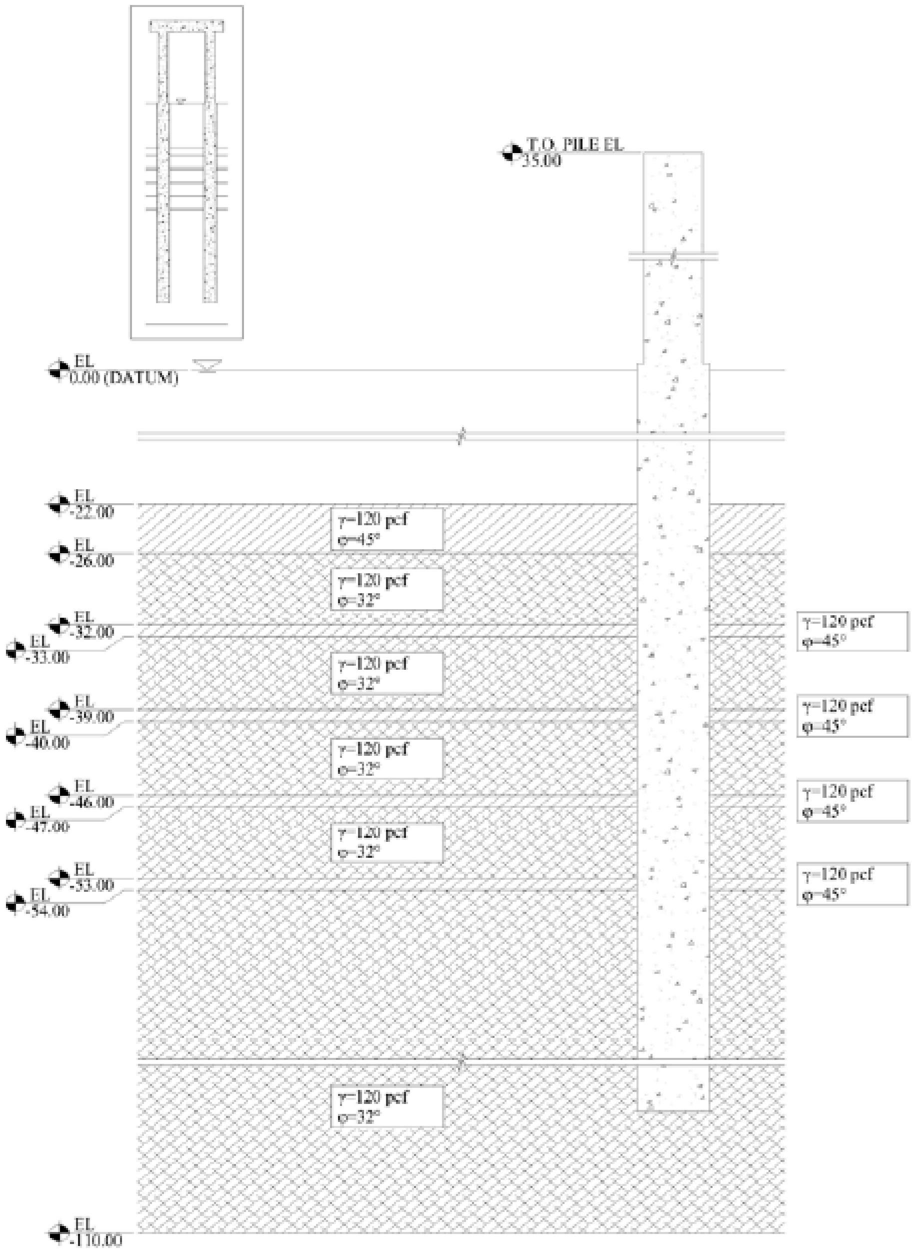

Figure 19. Pitt County bridge MultiPier model for a single drilled shaft and bridge bent (inset).

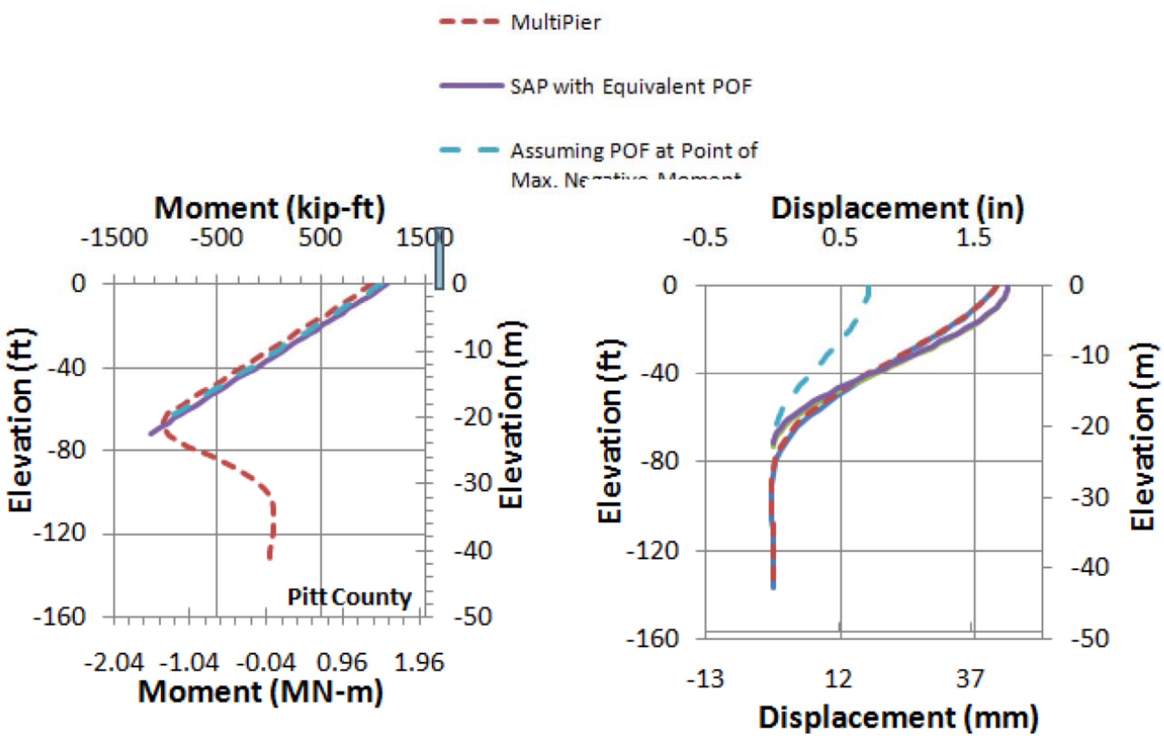

Figure 20. Distribution of Bending Moment and Lateral Deformation with Depth for a Single Pile-Pitt County

Table 13. Pitt County Nonlinear Analysis Results, Top of Shaft: Maximum Lateral Load Case

Model	Maximum Moment (kip-ft)	Maximum Axial Force (kips)	Maximum Shear Force (kips)
SAP—Nonlinear	978	832	35.7
MultiPier	1098	807	33.3
SAP—Equivalent	1245	790	35.4
AASHTO Group	AASHTO Group II Shaft 2, Demand Capacity Ratio = 0.302		

Optimization of Design

The model was then optimized by reducing the number or size of the shafts while maintaining the same load cases. For this bridge, two 60 in. diameter drilled shafts were used. Table 14 shows the results obtained by reducing the number of shafts. DCRs tend to vary greatly, approaching as high a value as 0.7 for the 48 in. shafts.

Table 14. Pitt County Alternative Shaft Configurations, based on AASHTO Groups I, IA, II, III and IV

	Two 60 in. shafts, 54 in. columns	Two 54 in. shafts uniform	Two 48 in. shafts uniform
Demand/Capacity Ratio (Shafts)	0.308	0.386	0.70
Displacement, transverse (shaft top, in)	1.3	1.7	2.6
Displacement, longitudinal (shaft top, in)	2.1	3.6	8.4
Displacement, axial (shaft top, in)	0.7	0.7	1.0

In this case, the model for the original 60 in. diameter shaft layout resulted in longitudinal displacements of approximately two inches. It should be noted, however, that the worst case soil profile was used, where maximum scour was assumed. With that in mind, the analysis based on the modeling configuration yielded displacements that are significantly greater than the normal 1 in. limit, especially in the longitudinal direction.

RECOMMENDED PROCEDURE FOR COMPUTING THE EQUIVALENT MODEL PARAMETERS:

1. Perform a nonlinear lateral single shaft analysis. In the analysis, representative properties of the soil and pile shall be used. If some properties/conditions differ from shaft to shaft within the bent (i.e. different P-y multipliers to account for group effects, rotation of local axes in piles, battered piles), the designer shall average the properties so the response of the single shaft is roughly representative of the bent. Alternatively, the worst case scenario can be used.
2. Apply boundary conditions at the top of the shaft (i.e. free-head or fixed head condition) as appropriate and apply the expected axial and lateral load under each condition.

3. Run the nonlinear analysis for the free and fixed head condition. Get the moment and displacement profiles for the single shaft, and determine the top displacement (δ), the maximum moment (Mmax) and the location of inflection points.
4. Compute the equivalent model parameters using Equations 1 through 4.
5. Build, analyze, and design an elastic frame model using the geometry, section properties and calculated equivalent model parameters. This might require the use of frame analysis software such as RCPier or others.

SUMMARY AND CONCLUSIONS

The work presented in this paper aims at understanding and optimizing the design process of drilled shafts bents for safety and functionality. The work includes the examination of the design process for drilled shaft bents and the process used to designate a corresponding point of fixity which is commonly used to estimate the shaft length for linear frame analysis. Modeling of three bridge structures was performed within the framework of MultiPier, SAP 2000, and SAP 2000 with the use of POF approach. The modeling was used to characterize the impact of the current assumptions on sizing the various components of the bridge bent. The results demonstrated the following:

i. MultiPier model results can be reproduced in the 3D SAP program, which verifies the results from MultiPier.

ii. The equivalent frame model proposed in Robinson et al. (2006) provides results that are comparable to those obtained from both the SAP and MultiPier analyses, given that the most critical lateral load case is used to evaluate the parameters for the equivalent model. The equivalent frame model yields similar moments, axial loads and shear loads in the most critical case. These findings should lead to more optimal, and possibly reduced, sizing of the structural elements.

iii. While the use of POF approach based on the point of maximum negative moment show similar maximum moment to values obtained from Multipier, the deformation was consistently underpredicted due to not accounting for inertia reduction (α) to match the nonlinear stiffness used in the MultiPier software.

iv. Reducing the number or size of the shafts while maintaining the same load indicates the feasibility of optimizing the design. For example, in the Wake County bridge case, seven 54 in. diameter drilled shafts were originally used as the foundation support system. Analysis results indicate the possibility of optimized design by reducing the size of the shafts to 30 in. while maintaining DCRs of less than 0.5. Lateral displacements in the transverse direction exceeded 2.5 inches. Further reduction of the shaft diameter was not possible, as the analysis would not converge. If the superstructure elements (including the bearing pad connection) could accept lateral displacements of this magnitude, and no further extreme events are expected, then this design outcome appears to be valid.

v. In all three case studies, some savings in material and installation costs can be realized using the nonlinear bent-soil analysis. Thus, compared to the point-of-fixity methods traditionally used, there is some room for cost and material savings by using the equivalent model proposed by Robinson et al. (2006).

Work in this paper provides a better understanding of the performance of bridge bents supported by drilled shafts under AASHTO loading conditions. Such understanding serves as a powerful tool that provides the flexibility of specifying the level of conservatism to be built into a specific bridge bent design.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the efforts the North Carolina Department of Transportation (NCDOT) Structures and Geotechnical Units for funding this research. The technical contributions of NCDOT engineers throughout the course of the work were invaluable. Their help and assistance are highly valued and appreciated.

REFERENCES

American Association of State Highway and Transportation Officials (AASHTO). (2004). AASHTO LRFD Bridge Design Specifications, Customary U.S. Units, Third Edition, Washington, D.C.

Bridge Software Institute (BSI). (2004). FB-MultiPier Manual, Ver. 4. Univ. of Florida.

Chen, Y. (1997). "Assessment of Pile Effective Lengths and Their Effects on Design—I. Assessment," *Computers and Structures*, Vol. 62, No. 2. 265-286.

Computers and Structures, Inc. (2003). SAP Manual. Berkeley, California, USA.

Davisson, M.T., and Robinson, K.E. (1965). "Bending and Buckling of Partially Embedded Piles." *Proc. Sixth International Conference Soil Mechanics and Foundation Engineering.* University of Toronto Press, Montreal, Canada. 243-246.

Ensoft, Inc. (2008). LPILE Manual. Austin, Texas, USA

Matlock, H. (1970). "Correlations for Design of Laterally Loaded Piles in Soft Clay," Paper No. OTC 1204, *Proceedings, Second Annual Offshore Technology Conference*, Houston, Texas, Vol. 1,. 577-594.

Reese, L.C., Cox, W. R. and Koop, F. D . (1974). "Analysis of Laterally Loaded Piles in Sand," Paper No. OTC 2080, Proceedings, Fifth Annual Offshore Technology Conference, Houston, Texas, (GESA Report No. D-75-9).

Reese, L.C., and Welch, R.C. (1975). "Lateral Loading of Deep Foundations in Stiff Clay," Journal of the Geotechnical Engineering Division, ASCE, V 101, N 7, Jul, 1975, pp. 633-649.

Robinson, B., Suarez, V., Robalino, P., Kowalsky, M. and Gabr, M. (2006). "Pile Bent Design Criteria", NCDOT Research Project 2005-19, North Carolina Department of Transportation, Raleigh, NC.

Robinson, B., Suarez, V., Robalino, P., Kowalsky, M. and Gabr, M. (2007). "A Point of Fixity Model for Pile and Shaft Bents", *Geotechnical Special Publication*, n 158, *Proceedings of Sessions of Geo-Denver 2007 Congress: Contemporary Issues in Deep Foundations (GSP 158).*

CHAPTER 4: Pile Toe Displacement Models for Driven Pile Bent Design in FB-MultiPier

ABSTRACT: Two models for driven pile toe load-displacement behavior are currently implemented in FB-MultiPier, a bridge design software package. The hyperbolic model implemented is primarily governed by the pile size, ultimate end bearing force as calculated by limit state methods or measured from load test programs and the soil's initial shear modulus. An empirical non-linear model is governed by the pile size and ultimate end bearing force. A review of the range of initial shear moduli measured in soils and correlated to common *in situ* tests indicates a typical range of around 1 to 50 ksi, depending on soil type and strength. Case histories from the literature are developed to compare the performance of the hyperbolic and nonlinear models. Application of the case history soil information to the respective toe models would indicate the hyperbolic model tends to underpredict the stiffness behavior of the pile relative to the nonlinear model for the common range of shear modulus. This underprediction of pile toe stiffness can adversely affect the efficiency of the pile and cap design on predominantly end bearing piles supporting bridges.

INTRODUCTION

Much of the geotechnical research for Load Resistance Factor Design (LRFD) of highway bridge foundations in the United States took as its initial focus strength limit state design. As experience with the design method and calibration of strength limit state resistance factors by states and practitioners has improved, research focus is shifting to a better understanding of serviceability limit states and the methods used to calculate expected displacements under foundation loads. For driven pile deep foundations, one way to estimate displacements under axial loading is through so-called t-z and q-z (or q-w) analyses, which use nonlinear curves of unit resistance or resistance versus displacement in conjunction with a lumped mass model to calculate the pile top movement under a given applied load (Coyle and Reese, 1966). This method was initially implemented in program FB-Pier, which was supported by the Federal Highway Administration and Florida DOT, and continues to be used in its successor program FB-MultiPier (BSI, 2014).

PREVIOUS RESEARCH

This study follows work summarized by Robinson et al. (2011) and Robinson et al. (2012) on driven pile bent and drilled shaft bent foundations. The driven pile study initially used FB-MultiPier's built-in q-z model by McVay et al. (1989) for estimating displacement due to loading at the pile toe. Recommendations from BSI (2004) for correlating the required shear modulus to *in situ* soil investigation techniques such as the standard penetration test and the cone penetration test were initially used as estimates for the required shear modulus in the q-z model in the first study. However, it was observed that high relative vertical displacements between two deep foundation elements within a single bent generated large moments in the bent cap, particularly under the predominantly live load cases. Because the moments were much higher than those calculated by other methods in use by NCDOT at the time, and displacements were higher than observed in practice, the q-z models were altered in those studies. Accordingly, it was decided to "fix" the pile toe throughout the analysis presented in Robinson et al. (2011) for the predominantly end bearing pile. For those cases, a model was adopted using a load-displacement curve that increased linearly until a displacement of 0.1 inch (2.5 mm) is reached; then plastic failure occurs at 1000 kips (4450 kN).

McVay et al. (1989)'s initial description of the method relating pile toe response to end bearing indicates two initial correlations on 10.75 inch diameter closed end pipes at both the FHWA sand site at Hunter's Point California (DiMillio et al., 1987a) and at the FHWA clay site in Houston Texas (DiMillio, 1987b). For the sand site, the instrumentation in the piles indicated predominantly end bearing (78%), an initial shear modulus for the sands at the toe of 8.5 ksi from correlation to the cone penetration tip resistance, and end bearing of 86 kips (unit end bearing of 136 kips/ft^2). For the clay site the pile instrumentation indicated predominantly side shear (76%), initial shear modulus reportedly from cross hole shear wave velocity, and with end bearing of 40 kips (unit end bearing of 63 kips/ft^2). Correlations indicated in the original paper showed good agreement.

Zhang et al. (2008) used the FB-MultiPier nonlinear q-z and t-z curves in a study of H-Piles driven in Hong Kong, with the goal of comparing predicted settlements to settlements from static load test results at the applied design load and twice the design load. The H-piles were driven to predominantly weathered rock or competent rock strata with gradually increasing SPT N-values. Overall, predicted settlements (which include predicted ultimate shaft and toe resistance as well as correlated shear modulus) were underpredicted at the investigated loads, but the authors also noted that shaft resistance appeared to be overestimated while end bearing was underestimated. Thus, the bulk of the settlement prediction at these loads was influenced by the shaft resistance model.

AXIAL NONLINEAR (Q-Z), END BEARING MODELS IN FB-MULTIPIER

The McVay et al. (1989) end bearing model for driven piles built in to FB-MultiPier is shown in Equation 1.

$$z = \frac{Q_b(1-\nu)}{4r_oG_i\left[1-\frac{Q_b}{Q_f}\right]^2} \qquad \text{Eq 1}$$

where Q_b = End bearing force applied to toe, Q_f = Ultimate end bearing force as calculated by limit state methods, z = axial displacement, G_i = Initial (low strain) shear modulus, r_o = pile radius and ν = Poisson's ratio.

A second q-z model (API RP2A, 2003), defined in Table 15, was added (BSI, 2014) for open ended pipe piles can be used, per API, "in the absence of more definitive criteria…for both sands and clays." The final q-z model in FB-MultiPier is a custom model, which the designer should use if local experience or a pile load test program result is available. For example, as simple elastic-perfectly plastic model like that implemented for wave equation analysis (PDI, 2010) could be adopted, with the displacement corresponding to transition from elastic to plastic behavior set at the pile diameter divided by 60 for loose to dense sands.

Table 15. Nonlinear toe response from API, 2003.

$z/2r_o$	Q_b/Q_f
0.002	0.25
0.013	0.50
0.042	0.75
0.073	0.90
0.100	1.00
∞	1.00

As can be seen in Figure 21, the McVay et al. (1989) toe response is very sensitive to the value of initial shear modulus input. In Figure 21, an ultimate end bearing force of 640 kips (2850 kN) is input based on an 18 inch pile (450 mm) diameter and a unit end bearing of 360 kips/ft^2 from a limit state analysis such as Vesic (1977) or from a measured load in a static or dynamic load test. In most load test interpretation methods, *total* pile capacity, or the sum of the ultimate end bearing and shaft resistance forces, is sometimes defined as the capacity mobilized at displacements equivalent to 1 inch or 10% of the pile diameter or width. Most static end bearing prediction models tend to give ultimate resistances that are mobilized at displacements slightly higher than expected to be experienced by the structure.

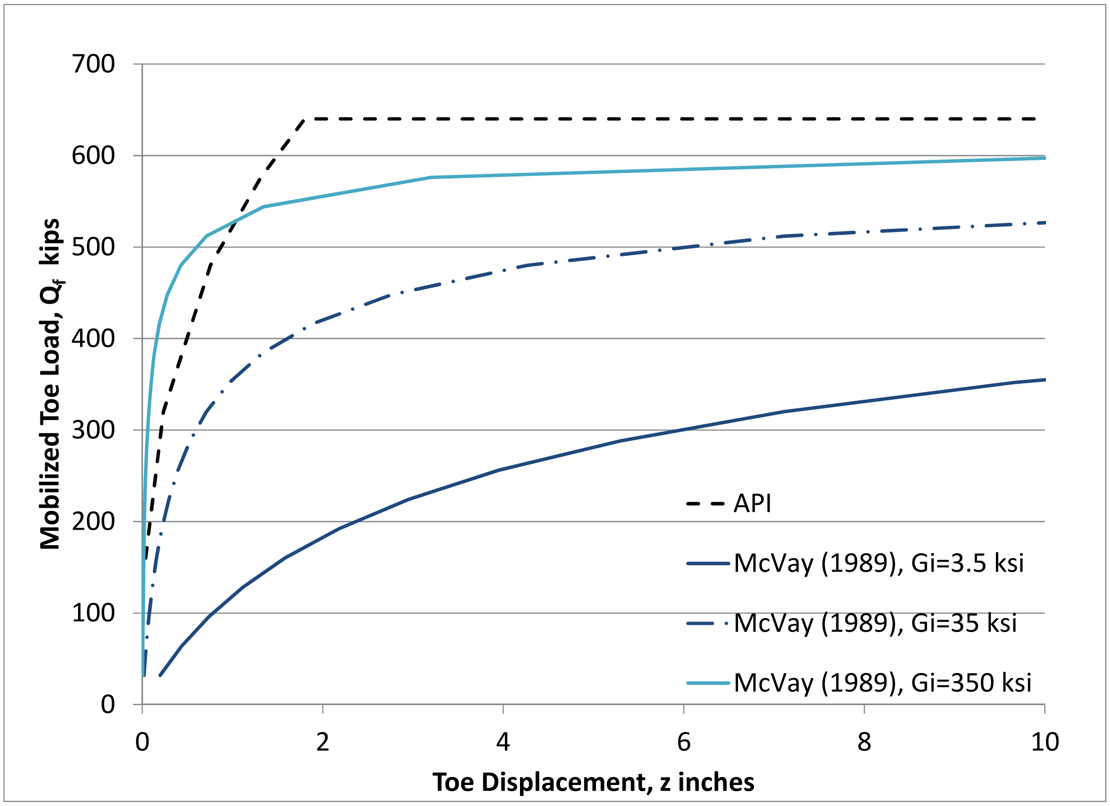

Figure 21. Comparison of non-linear toe models, assuming 18 inch diameter closed ended pipe

Using the standard static end bearing model parameter recommendations in FB-MultiPier creates a situation in which reaching the ultimate end bearing capacity requires very large displacements. As shown in Figure 21, to obtain 90% of the 640 kip ultimate capacity on an 18-inch diameter pipe with a secant shear modulus of 3.5 ksi (a value typical of soft clay, according to BSI, 2014), the model predicts that the pile must move more than 300 inches. At a shear modulus of 35 ksi (a value typical for dense sand), the pile must move 30 inches. Even at a very high initial shear modulus of 350 ksi (shown in Figure 21 for illustrative purposes), the pile must move 3 inches, or 17% of the modeled pile diameter, to mobilize 90% of this example's ultimate capacity.

Q_f

Initial Shear Modulus Values

The FB-MultiPier manual (BSI, 2014) suggests estimating shear modulus from the N-value per Imai and Tonouchi (1982) or via elastic modulus per Kulhawy and Mayne (1990) and Poisson's ratio. These are shown on Figure 22. For clays, the MultiPier manual suggests clay shear moduli in the range of 1 to 15 ksi, depending on overconsolidation ratio and plasticity index.

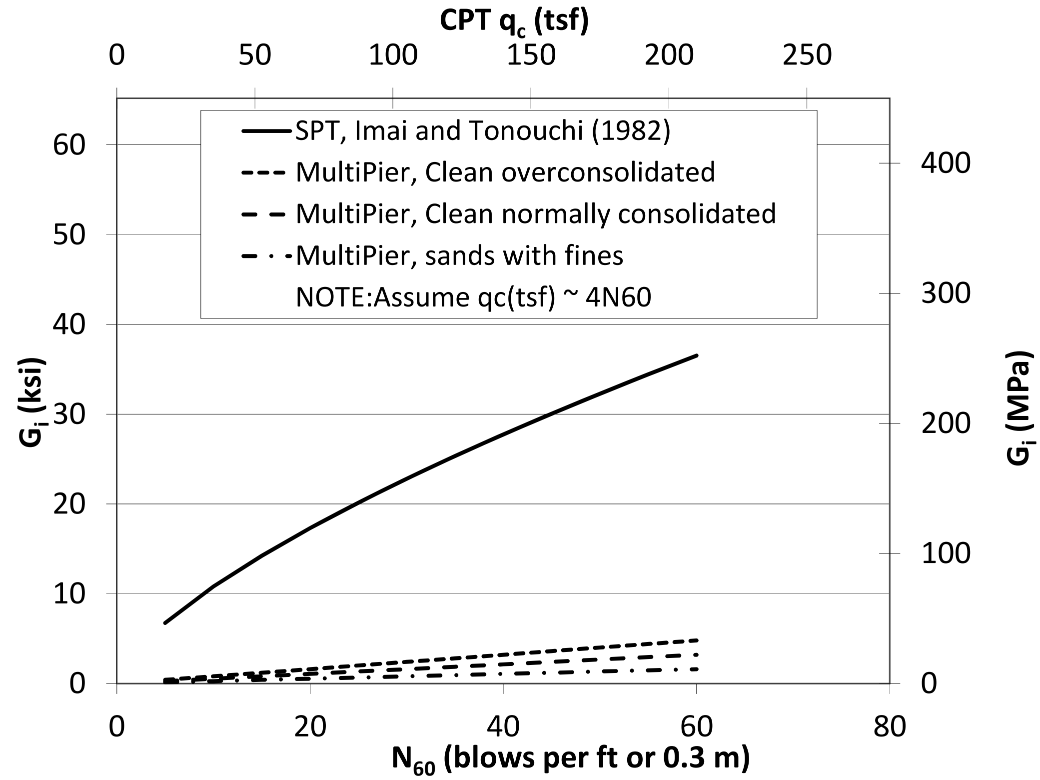

Figure 22. MultiPier suggested shear moduli curves from SPT N-Value, sands

McVay et al. (1989) also used a correlation on the Hunter's Point sand load test site between the shear modulus and Cone Penetrometer Test (CPT) tip resistance, after Robertson and Campanella (1984). Figure 23 shows the estimated shear modulus obtained from these sources as well as SPT correlations to initial shear modulus from Ohta and Goto (1978) and Rix and Stokoe (1991). The latter moduli were calculated assuming a pile penetration of 40 ft (12.2 m) and an effective unit weight of 120 lb/ft^3 (18.8 kN/m^3). These values are an order of magnitude higher than initially recommended in the program.

Figure 24 (Harden and Black 1968) and Figure 25 (Borden and Shao 1995) shows initial shear modulus measurements for sands and residual silty soils, respectively, from resonant column and torsional shear tests. Based on these correlations from several different sources, initial shear modulus in soils at depths typically associated with pile tip elevations appear to limited to approximately 60 ksi (414 MPa), with a more typical upper bound of around 35 ksi (207 MPa).

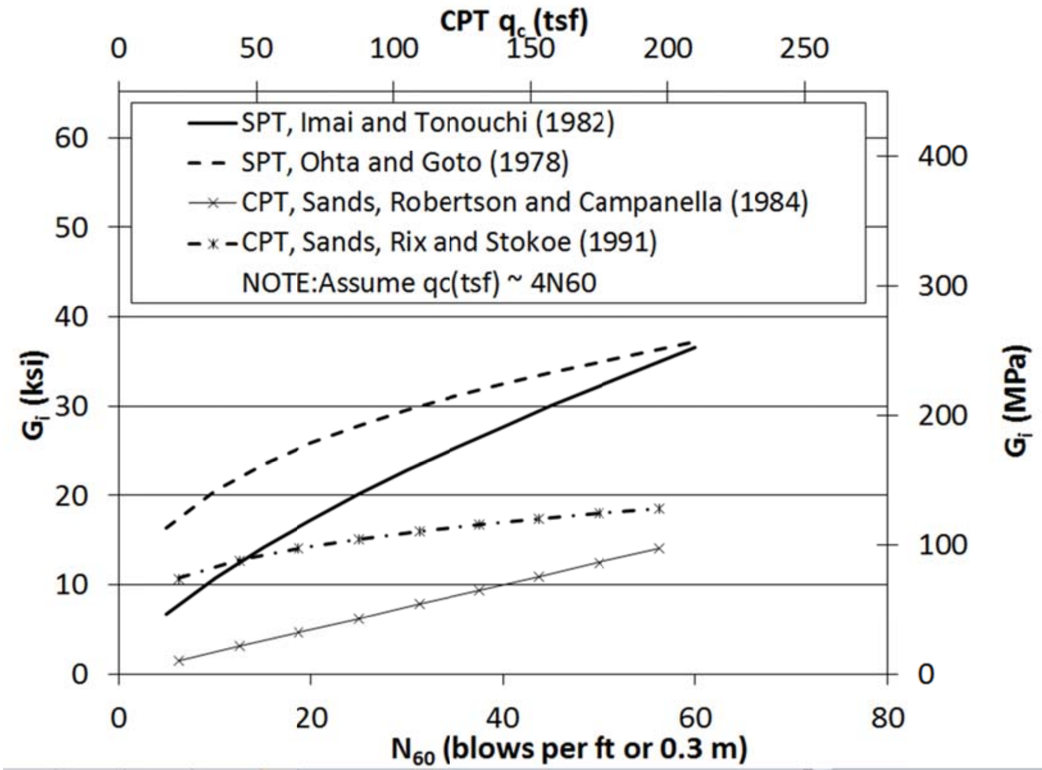

Figure 23. Low strain shear modulus correlations versus N-Value and cone tip resistance for sands

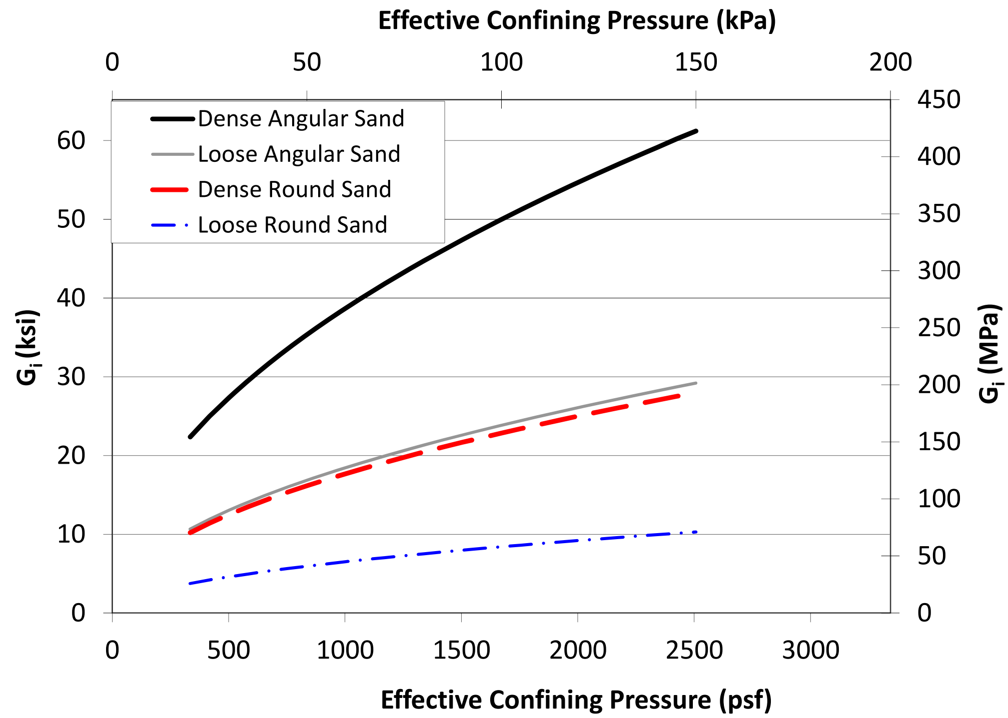

Figure 24. Initial shear modulus for sands after Harden and Black (1968)

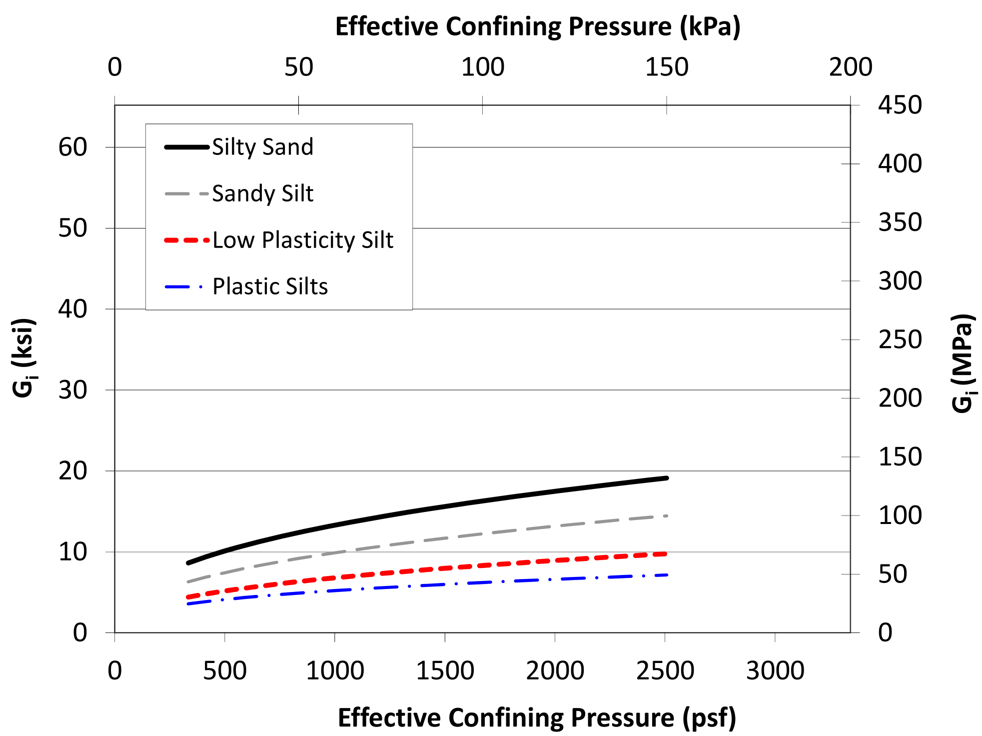

Figure 25. Initial Shear Modulus for silty residual materials after Borden and Shao 1995

CASE STUDIES

Closed End Pipe Pile--FHWA Hunter's Point, California

The data presented in McVay et al. (1989) for the single pile static load test from a 10.75 inch closed ended steel pipe pile as reported by DiMillio et al.(1987a) is revisited. This single pile and a group of piles were placed in hydraulically placed sand in California. The pile length was 30 feet. For the pile tip, the Poisson's ratio was assumed to be 0.30, the unit end bearing was based on the average cone tip resistance (q_c) one diameter above and below the pile tip elevation, and the initial shear modulus (G_i) was reportedly calculated as displayed on Figure 23 per Robertson and Campanella (1984) as

$$G_i(psi) = 125q_c(tsf) \quad \text{Eq 2}$$

The predicted tip resistance reported in the paper was 83 kips (corresponding to a unit end bearing or average cone tip resistance of 132 ksf or 66 tsf). This yields an initial shear modulus of 8,230 psi or 8.23 ksi. Substituting Q_f = 83 kips, G_i = 8.23 ksi, r_o = 5.375 inches and ν = 0.3 into Eq 1 yields the lowest dashed curve in Figure 26. Using Rix and Stokoe's correlation of initial shear modulus from Figure 23 yields a G_i of 16 ksi, while the dashed curve still under predicts the stiffness response. The API q-z curve is also shown for reference.

When the q-z curves were verified in MultiPier for this case, the average unit shaft resistance corresponding to the reported measured skin resistance of 24 kips (0.341 ksf unit skin friction) and the reported measured tip resistance of 86 kips (0.914 ksi unit end bearing) were used to recreate the overall load-deflection behavior of the single pile. Figure 26 indicates the API methods appear to provide a better match in this situation, despite the method's origins in larger diameter open ended pipe piles.

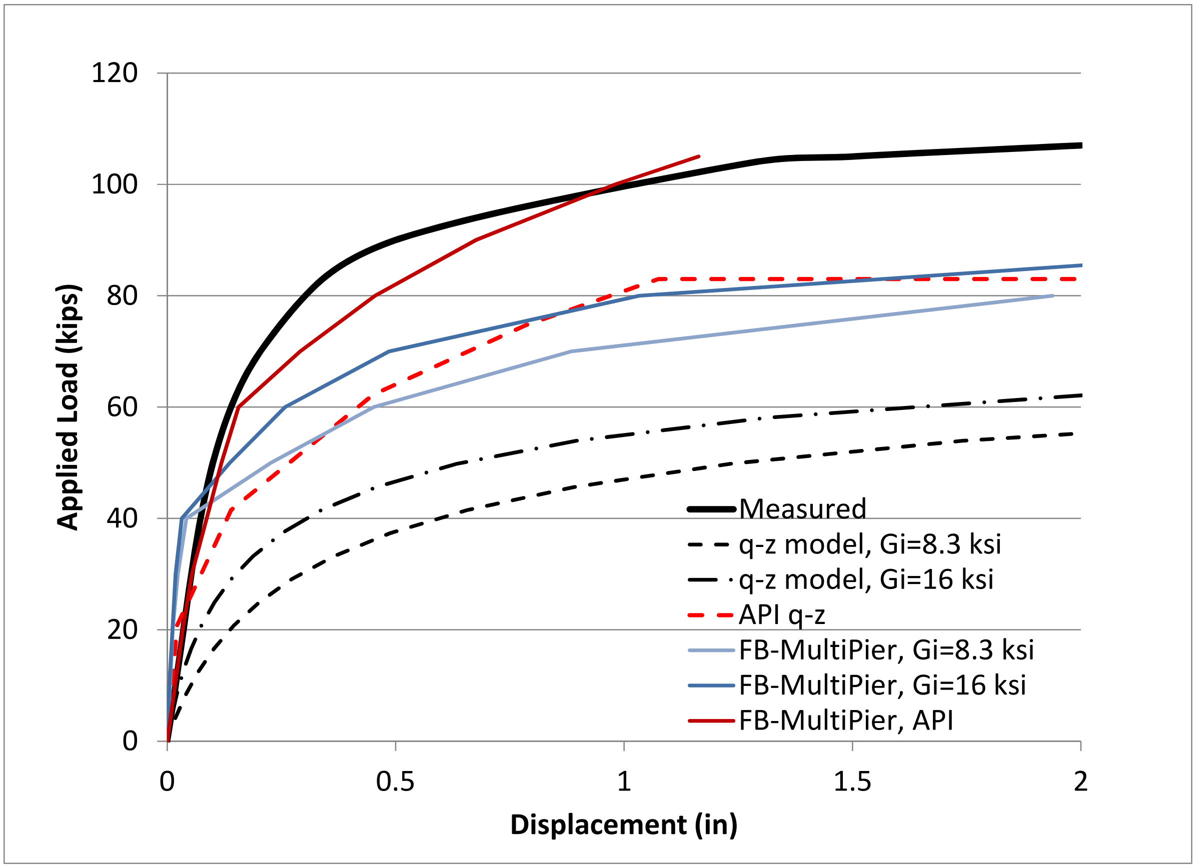

Figure 26. Hunter's Point Pile in Sand, with two CPT shear modulus models and the API model

H-Pile—Jacksonville, Illinois

Long (2002) describes a series of static load tests performed with high strain dynamic testing on a steel HP12x53 H-Pile. The static load test pile was installed to a penetration 30 feet (9.38 m), tested statically, then redriven to 38 feet (11.54 m) for a second set of load tests and restrikes. The initial installation was driven through silty clays and sands with N-Values between 5 and 10 blows per foot to a sandy dense till with N-values around the tip in excess of 50 blows per foot. The static load test failure Davisson load at 30 feet (9.38 m) was indicated to be 1202 kN (270 kips). Signal matching analyses with CAPWAP® (PDI, 2014) on restrike data from a high strain dynamic load tests performed immediately after the static load test estimated at total ultimate capacity of 284 kips (1263 kN) with 150 kips (667 kN)in shaft resistance and 134 kips (596 kN) in end bearing.

The N-Value at the tip presents a special problem. N-Values in excess of 50 blows per foot are generally not reported per ASTM D1586-11 (ASTM, 2011). An equivalent N-value exceeding 50 blows per foot, indicates the practical limit of the hammer/split spoon driving system more than the strength of the soil, and the correlations of initial shear modulus to N-value likely do not include large numbers of high N-value samples. The most likely shear modulus in this material is difficult to predict. The SPT methods in Figure 23 would indicate extrapolated initial shear moduli of 30 to 50 ksi at higher blow counts. Along the shaft, N-values of 5 to 10 blows per foot would indicate an initial shear modulus of 10 ksi. Poisson's ratio was assumed to be 0.3, and the signal matching estimates of shaft resistance and end bearing were used in Equation 1 and FB-MultiPier.

Figure 27 shows the comparison between the measured static load set curve and the computed curves from the hyperbolic model and the API models. While there is still a significant stiffness reduction on the hyperbolic model, the early portion of the curve is well captured.

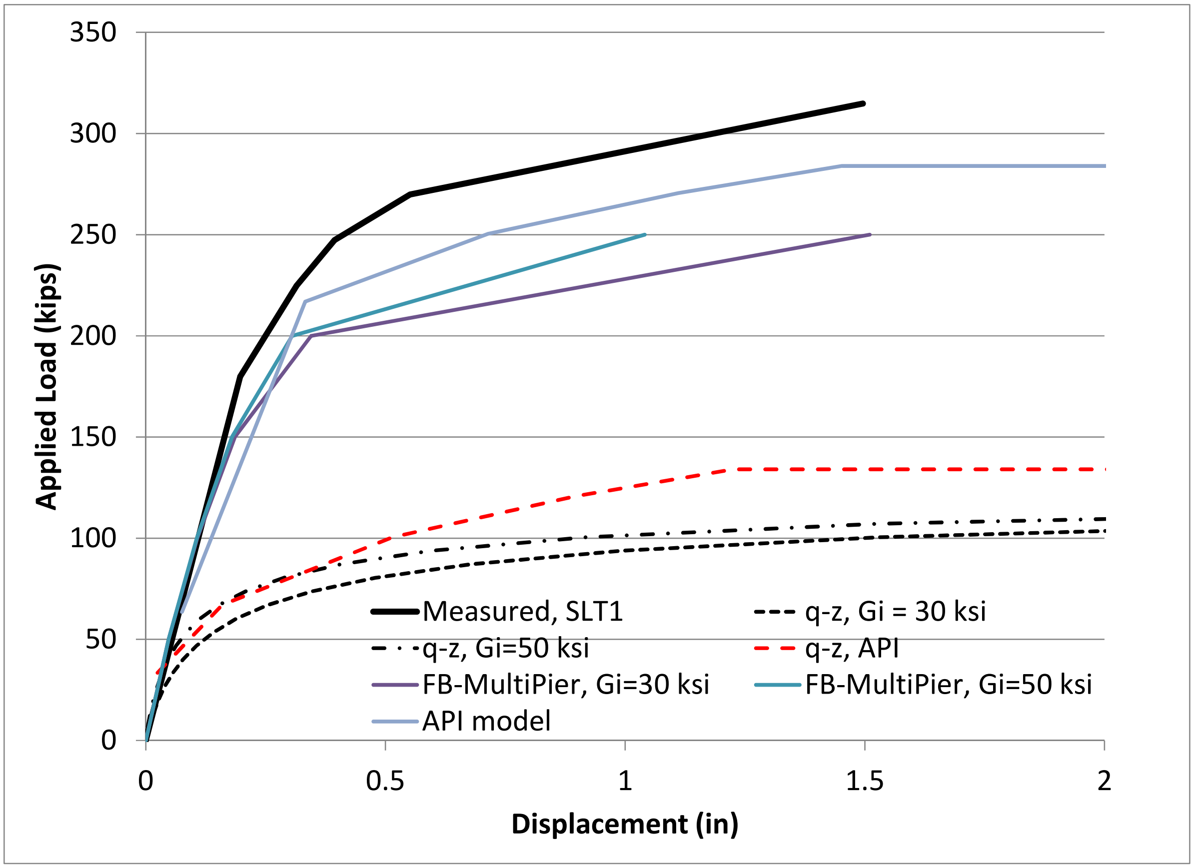

Figure 27. Jacksonville, Illinois H-Pile Measured versus computed curves

PILE BENT MODEL, REVISITED

In Chapter 2, the Northampton County model's piles were closed ended 24 inch pipe piles, installed with a final tip elevation in soils described as sandy clay with N-values of between 20 and 30 blows per foot. In that case, a purely fixed end bearing response was not as justifiable as for the weathered rock materials. A parametric study is reported in Table 16, with a fixed ultimate end bearing of 450 kips and Poisson's ratio of 0.3 and was performed using the hyperbolic and API toe models. The maximum axial force in a pile is quite similar across all analyses, as would be expected. Given the varying stiffness in the hyperbolic and API models, however, the same maximum axial force yields a sizable difference in maximum axial displacement. Using a lower initial shear modulus yields increases in the maximum cap beam moment of up to 32%. Practically, however, underpredicting the toe stiffness behavior would lead to piles potentially driven deeper and an overdesigned cap beam.

Based on the SPT N-values reported at the tip, correlations suggested in the FB-MultiPier manual would suggest a shear modulus of between 1 and 20 ksi, as shown in Table 2. If the pile tip were interpreted to be driven to very dense angular sand, there is relatively little difference between a very dense sand shear modulus (50 ksi) model and the API model, largely due to the size of the pipe. The 24 inch pipe pile would require a two inch displacement to fully mobilize the API curve, and given that the ultimate capacity of the pile toe has not yet been reached, the two models predict the same displacement, much like the H-Pile shown in Figure 7.

Table 16 . Northampton County Pipe piles, Tipped in Sand, Various models

Model	Maximum axial displacement (mm/in)	Axial pile force (kN/kips)	Maximum moment in cap beam (kN m/kip ft)
Fixed, previously reported	6.4/0.24	1570/353	865/637
Gi = 1 ksi	241/9.5	1423/320	1137/839
Gi = 10 ksi	33.8/1.33	1486/334	1036/764
Gi = 50 ksi	12.4/0.49	1525/343	926/683
API	13.7/0.54	1530/344	939/693

SUMMARY AND CONCLUSIONS

A comparison of the FB-MultiPier driven pile hyperbolic toe model for a commonly cited range of values of a soil's initial shear moduli indicates that excessive pile toe displacements are required to mobilize the ultimate pile toe capacity, as measured from load tests or calculated from geotechnical static analysis. Compared to the nonlinear toe response implemented by API, for end bearing piles driven in soil, the hyperbolic model would significantly overpredict displacements for most values of shear modulus. This overprediction of displacements could yield higher predicted differential displacements at the pile top, yielding higher bending moments predicted in the cap beam.

REFERENCES

American Petroleum Institute (1993). API RP 2A-LRFD. Dallas, Texas.

ASTM Standard D1586, (2011). Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils," ASTM International, West Conshohocken, PA, 2003, DOI: 10.1520/C0033-03

Borden, R.H. and Shao, L. (1995). "Construction Related Vibrations: Field Verification of Vibration Induced Settlement Model—Final Report." Report to the North Carolina Department of Transportation. 197 pages.

Bridge Software Institute (BSI). (2004). *FB-MultiPier*, Version 4 *Manual*. University of Florida.

Bridge Software Institute (BSI). (2014). *FB-MultiPier Version 4.19.3 Manual*,. University of Florida.

Coyle, H.M.; Reese, L.C. (1966) Load transfer for axially loaded piles in clay *American Society of Civil Engineers Proceedings, Journal of the Soil Mechanics and Foundations Division*, v 92, n SM2, p 1-26, Mar, 1966

DiMillio, AF, Ng, Es, Briaud, JL and O'Neil, MW. (1987a). "Pile Group Prediction Symposium: Summary Volume 1: Sandy Soil." FHWA Report FHWA-TS-87-221.

DiMillio, AF, Ng, Es, Briaud, JL and O'Neil, MW. (1987b). "Pile Group Prediction Symposium: Summary Volume 2: Clay Soil." FHWA Report FHWA-TS-87-222.

Hardin, B.O. and Black, W.L. (1968), "Vibration Modulus of Normally Consolidated Clay," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 94, No. SM2, Proc. Paper 5833, March, 353-368.

Imai, Tsuneo (1982). "CORRELATION OF N VALUE WITH S-WAVE VELOCITY AND SHEAR MODULUS.". Penetration Testing, Proceedings of the 2nd European Symposium. Volume 2: Cone Penetration Test., p. 67.

Long, J.H. (2002), "Resistance factors for driven piling developed from load-test databases" *Geotechnical Special Publication*, n 116 II, p 944-960.

McVay, M.C., Townsend, F.C., Bloomquist, D.G. and Caliendo, J.A. (1989). "Numerical Analysis of Vertically Loaded Pile Groups." Foundation Engineering: Current Principles and Practices. *Proceedings of the Congress. Edited by F. H. Kulhawy, American Society of Civil Engineers.* 675-690.

Ohta, Y. and Goto, N. (1978), Empirical shear wave velocity equations in terms of characteristic soil indexes. Earthquake Engng. Struct. Dyn., 6: 167–187.

Pile Dynamics, Inc.,(2014). CAPWAP® Background Report. Cleveland, Ohio

Rix, G.J. and Stokoe, K.H. (1991). "Correlation of initial tangent modulus and cone penetration resistance" International Symposium on Calibration Chamber Testing, 351-362.

Robertson, P.K. and Campanella, R.G. (1984). "Guideline for Use and Interpretation of the Electronic Cone Penetration Test." Hogentogler & Co., 160 pages.

Robinson, B., Suarez, V.; Gabr, M. A.; Kowalsky, M. (2011) "Simplified lateral analysis of deep foundation supported bridge bents: Driven pile case studies" *Journal of Bridge Engineering*, v 16, n 4, p 558-569.

Robinson, B, Suarez, V; Gabr, M; Kowalsky, M (2012). "Configuration optimization of drilled shafts supporting bridge structures: Three case studies" *Practice Periodical on Structural Design and Construction*, v 17, n 3, p 93-101.

Vesic, A.S. (1977). *Design of Pile Foundations*, NCHRP, Synthesis of Highway Practice #42, Transportation Research Board, National Research Council, Washington

Zhang, L.M., Xu, Y., and Tang, W.H. (2008). "Calibration of models for pile settlement analysis using 64 field load tests." *Canadian Geotechnical Journal,* v 45, p 59-73.

CHAPTER 5: Conclusions and Future Research

The work presented in this paper aims at understanding and optimizing the design process of drilled shafts bents for safety and functionality. The work includes the examination of the design process for drilled shaft bents and the process used to designate a corresponding point of fixity which is commonly used to estimate the shaft length for linear frame analysis. Modeling of three bridge structures was performed within the framework of MultiPier, SAP 2000, and SAP 2000 with the use of POF approach. The modeling was used to characterize the impact of the current assumptions on sizing the various components of the bridge bent.

Six bridges representing four pile types were selected to evaluate the current design practice that accounts for the lateral resistance of soil by introducing the equivalent length or point of fixity and modeling the bridge structure as an elastic frame. The models were developed using nonlinear pile sections, elastic-plastic bent caps (using cracked moments of inertia from moment-curvature analyses), and soil along the length of each pile modeled by P-y, t-z and q-z springs.

These bridges were modeled in both SAP and FB-MultiPier programs for comparative purposes. Several factors regarding the applicability of the nonlinear models and the representation of the soil and structural elements have been presented including the toe model in FB-MultiPier and the soil stiffness parameters with corresponding magnitudes of initial shear modulus and nonlinearity of reinforced concrete elements.

Results from MultiPier single pile analyses were used to develop the equivalent model parameters. For each of the three bridges, sensitivity of the model to variations in axial and lateral load were explored, and the results of the two nonlinear analyses, the equivalent frame model and a point of fixity approach used in practice were compared.

Based on the results of this study, the following conclusions are advanced for the driven pile bents:

i. Overall, data from the three bridge models show good agreement between the results obtained from FB-MultiPier versus SAP for the analyzed load cases; these findings serves to verify the FB-MultiPier results and provide a better understanding of the behavior of free-standing pile bents in general.
ii. Although it was not a significant issue for the three bridges investigated, differential axial deformation of the piles can play a role in moment demand in both the pile and the cap beam
iii. The results presented herein show that the equivalent frame model proposed in Robinson et al. (2006) provides results that are comparable to those obtained from both the SAP and FB-MultiPier analyses, provided that the most critical lateral load case is used to evaluate the parameters for the equivalent model.
iv. The equivalent frame model parameters are particularly sensitive to the comparable selection of both axial and lateral loads. If lateral loads used to develop the equivalent model are higher than experienced, the axial and lateral deflections, and moments will also be higher. For design purposes this is conservative.
v. The assumption of pile heads free to rotate in the bridge's longitudinal direction yields higher longitudinal displacements (and thus lower inertia reduction factors in the equivalent model). If the bridge designer requires calculation of and limits to this displacement, the assumption of a fully free pile heads should be revisited. The connection between sub- and super-structure might yield enough rotational stiffness to allow some partial fixity of the pile top.

The results from the drilled shaft bent modeling demonstrated the following:

i. MultiPier model results can be reproduced in the 3D SAP program, which verifies the results from MultiPier.
ii. The equivalent frame model proposed provides results that are comparable to those obtained from both the SAP and MultiPier analyses, given that the most critical lateral load case is used to evaluate the parameters for the equivalent model.

iii. The equivalent frame model yields similar moments, axial loads and shear loads in the most critical case. These findings should lead to more optimal, and possibly reduced, sizing of the structural elements.

iv. While the use of equivalent length approach based on the point of maximum negative moment show similar maximum moment to values obtained from Multipier, the deformation was consistently underpredicted due to not accounting for inertia reduction to match the nonlinear stiffness used in the MultiPier software. For service limit state considerations, the inertia reduction will be required.

v. Reducing the number or size of the shafts while maintaining the same load indicates the feasibility of optimizing the design. If the superstructure elements (including the bearing pad connection) could accept lateral displacements larger than 1 inch, and no further extreme events are expected, then these optimized design outcomes appear to be valid.

vi. In all three case studies, some savings in material and installation costs can be realized using the nonlinear bent-soil analysis. Thus, compared to the point-of-fixity methods traditionally used, there is some room for cost and material savings by using the equivalent model proposed by Robinson et al. (2006).

The results from the pile tip load-deformation studies indicated:

i. A comparison of the FB-MultiPier driven pile hyperbolic toe model for a commonly cited range of values of a soil's initial shear moduli indicates that excessive pile toe displacements are required to mobilize the ultimate pile toe capacity, as measured from load tests or calculated from geotechnical static analysis.

ii. Compared to the nonlinear toe response implemented by API, for end bearing piles driven in soil, the hyperbolic model would significantly over predict displacements for most values of shear modulus.

iii. An over prediction of toe displacements could yield higher predicted differential displacements at the pile tops, yielding higher bending moments predicted in the cap beam.

FUTURE RESEARCH

The preceding work lays the groundwork for a series of potential studies and refinements to the proposed method.

i. **Using free or pinned pile heads in the equivalent model.**
In most cases presented in these studies, the free to rotate and translate pile top used to determine the equivalent length in the longitudinal direction yielded the highest displacements Vidot-Vega et al. (2009) indicated elastomeric bearing pads acting as a connection between the superstructure and the substructure provided a level of rotation stiffness between a pile head that is free and fixed against rotation. Further investigation into the rotational stiffness of such connections could also improve the deflection calculations performed for bridge design.

ii. **Drilled shaft equivalent lengths and rock sockets.** There is still some discussion ongoing, such as Lien (2013) to try to alleviate the confusion between the geotechnical and structural indications of points of fixity or equivalent lengths, particularly in drilled shafts and the calculation of moments and shear forces at the rock socket. This is a useful and necessary discussion, which should be investigated and clarified as required.

iii. **Foundation optimization**. A move to service limit state design will also by necessity force structural and geotechnical engineers to re-examine the tolerable vertical and horizontal movements under a variety of load cases. This may expand the opportunities to optimize and reduce the sizes or number of foundations. However, this will also require a renewed review of the effects of redundancy on this optimization and on the ability of our geotechnical models to accurate predict structural displacements from *in situ* or laboratory tests of a soil layer's stiffness and consolidation behavior.

iv. **Service limit state stiffness model verification and calibration.** As stated in iii), a comprehensive review and verification of the available models to calculate the deformation due to loading will soon be required. For deep foundations, this will include shaft resistance, end bearing, and lateral resistance models. Foundations advanced into rocks and soft rocks will also require significant attention, and for non-bent bridges the load-deformation behaviors of continuous flight auger piles will require more careful scrutiny. While some emerging technologies, such as thermal integrity profiling, borehole scanning, and downhole inspection devices, are beginning to be applied, the engineer will be required to know the shape of cast in place drilled piles, as that will affect both their strength limit state behavior and their service limit state behavior.

REFERENCES

Lien, B. (2013). “AASHTO Methodologies for Drilled Shafts Supporting Bridges and Sign Sructures:” Presentation at the 7th Geotechnical Geophysical and Geoenvironmental Technology Transfer (Geo3T2) Conference. April 4 and 5, 2013.

Vidot-Vega, A.L., Possiel, B., Robinson, B., Kowalsky, M.J, and Gabr, M. (2009). "Evaluation of Rotational Stiffness of Elastomeric Bearing Pad-Anchor Bolt Connections on Deep Foundation Bents", ASCE Journal of Structural Engineering, V. 14, No. 6, December.