PILE BENT DESIGN CRITERIA

Project 2005-19

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16. Abstract

This study was undertaken to review the design of pile bents by the North Carolina Department of Transportation (NCDOT). Research was performed to investigate current design methodologies and compare practice approach to other feasible and cost effective design methods. As such, two goals were pursued: i) enhancing the current equivalent frame model of the pile bents such that it fully considers the resistance of the soil around the piles, and, ii) adopting a comprehensive state of practice, which involves the integration of a comprehensive software and analytical solutions. It was also deemed important to study displacement limits imposed on bridge design and suggest improved limits based on the results from the soil/structural models. To achieve the study objectives, four bridge case studies provided by the NCDOT were modeled. The 3-D finite element modeling utilized two software packages: MultiPier and SAP 2000. MultiPier was used as a tool to specifically analyze piles and bridge bents. SAP2000 was used to verify the MultiPier model, and as a tool to evaluate proposed equivalent frame models. Results indicated that MultiPier is the most promising program, considering future NCDOT applications. This is mostly due to the built-in soil models and packaged features that include AASHTO load cases and load resistance factors modules. If, however, MultiPier is to be used as a design tool, some preprocessing would be needed to determine live loads. Postprocessors are also required for design of bent cap section reinforcement. The results from the 3-D verification analyses performed in SAP showed good agreement with the MultiPier results. Bridge models built to optimize the design (that is, to reduce the size of or number of the piles used as well as the cap section's rebar and size) showed that cost savings may be realized using the nonlinear analysis approach. In order to achieve the objective of enhancing the current equivalent frame models, a method is proposed to calculate a Point of Fixity (POF) and equivalent frame height to better estimate moments, shears and displacements in bridge substructures. Based on this model, it is shown that NCDOT's current definition of POF is conservative. The proposed model yields results that are closer to results from nonlinear numerical analysis. Geotechnical and structural displacement limit states were investigated but it is clear that there is a lack in literature of well-defined limit states for both the super and sub structures. Preliminary limit state models are proposed to investigate allowable displacements due to opening of a gap between soil and pile and closing of expansion joints.

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EXECUTIVE SUMMARY

This study was undertaken to review the design of pile bents by the North Carolina Department of Transportation (NCDOT). Research was performed to investigate current design methodologies and compare practice approach to other feasible and cost effective design methods. As such, two goals were laid out: i) enhancing the current equivalent frame model of the pile bent such that it fully considers the resistance of the soil around the piles, and, ii) adopting a comprehensive state of practice, which involves the integration of comprehensive software and analytical solutions. It was also deemed important to study displacement limits imposed on bridge design and suggest improved limits based on the results from the soil/structural models.

In order to propose a modified state of practice and to compare current equivalent frame models with more comprehensive nonlinear analytical models, four bridge case studies provided by the NCDOT were modeled. The 3-D modeling utilized two software packages. MultiPier was used as an analytical tool specifically designed for the analysis of piles and bridge bents. SAP2000 was used to verify the MultiPier model, and as a tool to evaluate proposed equivalent frame models.

Results indicated that MultiPier is the most promising program, considering future NCDOT applications. This is mostly due to the built-in soil models and packaged features that include AASHTO load cases and load resistance factors modules. If, however, MultiPier is to be used as a design tool, some preprocessing would be needed to determine live loads. Postprocessors are also required for design of bent cap section reinforcement. The results from the 3-D verification analyses performed in SAP showed good agreement with the MultiPier results. Bridge models built to optimize the designs (that is, to reduce the size of or number of the piles used as well as the cap section's rebar and size) showed that cost savings may be realized using the nonlinear analysis methods.

To enhance the current equivalent frame models, a proposed method is presented to calculate a Point of Fixity (POF) and equivalent frame height to better estimate moments, shears and displacements in bridge substructures. Based on this model, it is shown that the NCDOT POF definition is conservative. The new model yields results that are closer to results from nonlinear comprehensive analysis than the current practice.

Geotechnical and structural displacement limit states were investigated. There is a lack in literature of clearly defined limit states for both the super and sub structures.

Preliminary models are proposed to investigate allowable displacements due to opening of a gap between soil and pile and closing of expansion joints.

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CHAPTER 1: INTRODUCTION

This one-year study was undertaken to review the design of pile bents by the North Carolina Department of Transportation (NCDOT). Pile bents are constructed using several driven piles that are connected at the top by a concrete cap beam. The bridge superstructure is most often connected to the pile bent through elastomeric bearing pads although other types of bearings are sometimes used. Loads from wind, traffic, the weight of the bridge and other sources are transferred from the superstructure into the subsurface bearing layers through the pile bent.

This project seeks to investigate design methodologies used by the NCDOT and to compare their current approach to other feasible design methods. Specifically, project work aims to address the following: i) improving the current state of practiceby enhancing an equivalent frame model of the pile bent to better model the contribution of the foundation soils, and, ii) adopting an improved state of practice, which involves the application of comprehensive software and analytical solutions. It was also deemed important to study displacement limits imposed on bridge design and suggest improved limits based on the results from the soil/structural models.

Objectives

The main objective of this study was to develop improved design/analysis guidelines for pile bents supporting bridge foundation. The study was motivated by NCDOT engineers who concluded that the analysis approach currently employed for design of pile bents maybe improved by investigating the level of conservatism built into it. Factors for consideration included defining the point of fixity for the piles under lateral loading, evaluating the extent of fixation of the pile top embedded in the pile cap, investigating the definition of the buckling k-factors, and proposing an approach for the establishment of allowable deformation criteria describing limit states. Specifically, the objectives of this study were as follows:

1. Investigation of existing 2-D linear frame model and the possibility of improving the design such that the extent of conservatism is quantified,

2. Performance of 3-D finite element analyses to include the soil surrounding the piles and any pile group action for the establishment of integrated substructure and super structure response,

3. Application of comprehensive design approach to four bridge case studies with various configurations, and compare the 3-D model results with the approach currently used in practice. The bridge case studies were selected to capture a variety of pile types, soil conditions and superstructure types,

4. Development of an equivalent linear model for improved analysis and design based on the results obtained from the numerical and analytical investigations, and,

5. Development of a framework to define super and substructure deformation limit states and the impact of the level of deformation on design outcome.

Scope of Work

This project plan was accomplished through a combined structural and geotechnical effort as the scope encompasses an issue of soil-structure interaction. The scope also required both detailed 3-D finite element analysis, and simplified frame analysis, for assessment of design and analysis recommendations. The five interrelated tasks pursued to accomplish the research objectives are as follows:

Task 1: Review of current NCDOT design practices, and other state of the art approaches. This first task was essential, as a detailed understanding of the current design and analysis procedure is fundamental to providing NCDOT with a useful product. While the team has an understanding of the assumptions made by both geotechnical and structural engineering units, the method by which those assumptions are implemented into practice had to be reviewed. The success of this task was greatly assisted by conversations and regular contact with NCDOT structural and geotechnical engineers. A summary of the findings from the review of the current NCDOT design practices is included in Chapter 2 of this report.

Task 2: Development of detailed 3D analysis models of pile bents to be used as benchmarks for comparison and development of design specifications for current practice. It is essential to have an understanding of how the pile-bents and bridge structure behave under the various applied loading conditions with and without extended simplifying assumptions and idealizations. The robust 3D analytical models developed in this task allowed the researchers to investigate impact of design factors including soil properties, pile cross section type and design details, pile length, group action, pile to cap connection, cap-beam flexibility, and superstructure contributions to the computed responses. The 3D analysis models are presented and evaluated in Chapter 3.

Task 3: Comparison of model results with current practice and development of model recommendations. For the configurations considered in the analysis models, current NCDOT practice was employed and the outcomes tabulated. By comparing the results in terms of forces, displacements, shear and moment distributions, the research team reconciled differences between the various models, and made recommendations regarding the appropriate analysis approach/parameters such that current design approach may provide results that are consistent with those predicted by the robust analysis model. Target parameters included depth to fixity and pile head rotation.

The model results are compared to current practice in Chapter 4. The comparative analyses used four bridge case studies, each configured differently and provided by NCDOT. Possible improvements to current practice (i.e. frame analyses) are discussed in Chapter 5.

Task 4: Recommendations for pile-bent performance limit states. At times, NCDOT practice utilizes a lateral displacement limit of one inch to as a deflection limit to assess the pile length. Utilizing the analytical model developed in task 2 and using other work reported in the literature, the development of a more rigorous guideline will be investigated Limit states for pile bent performance are evaluated in Chapter 6.

Task 5: Development of recommendations for rigorous analysis and design approaches. Improved design and analysis techniques were proposed. A series of conclusions and possible design procedures for current and future applications are suggested. These recommendations and conclusions can be found in Chapter 7.

CHAPTER 2: LITERATURE REVIEW AND BACKGROUND

Literature review—Displacement Limit States

Most of the literature concerning displacements of bridge structures takes into account aspects related to the relationship between loads and displacements without specifying well defined limits. In cases where there are some defined limits, they are general. There are also some references in the literature about bridge approach embankments and bridge foundation movements.

Moulton (1986) surveyed field reports on the condition of 314 bridges from 39 states. In this study, damage to or loss of use of some part of the bridge structure was recorded, as was the location and direction of any observed movement of the piers or abutments. Based on a close examination of this data base, Moulton made a few general observations which are summarized in Table 1. These observations identify trends in development of damage—some bridges in the study had movements in excess of those summarized in the table, and yet the movement was deemed tolerable.

Table 1. Moulton's Movement Limits for Bridges

Direction of Movement	**Magnitude most likely to cause intolerable damage**
Vertical Only	4 inches
Horizontal (Lateral) Only	2 inches
Both Horizontal and Vertical	
Vertical Component	2 inches
Horizontal Component	1 inch
Angular Distortion (Differential Vertical Displacement : Span Length)	0.004
Multispan structures have a higher frequency of severe structural damage due to foundation movements than single span bridges.	

The movements presented in Table 1 are gross movements of an abutment or pier. They are not due to a particular loading condition or do not discuss differential settlements due to uneven loading across a single pier. Most of the movements discussed in the report seem partly due to long-term settlement of either the abutment or pier foundations or due to long term settlement of the approach embankments. In this current study, the displacements reported in Chapter 4 and 5 and both the SAP and MultiPier programs do not consider possible long term settlement of the soil layers around and under the piles. This geotechnical analysis is likely most critical for the abutment fills and foundations, as the pile elements in driven pile bents are generally driven to partially weathered rock or a stiff bearing strata that would not be subject to consolidation type settlement.

Moulton's work has been used elsewhere, as others usually refer to the numbers in Table 2.1. Zhang *et al.* (2002) used MultiPier to model pile groups under axial dead loads, and verified the results with centrifuge modeling. The authors used the two inch horizontal displacement limit as suggested by Moulton, as a basis for comparison for group pile displacements under various axial loads.

Liu et al. (2001) considered three limit states in their pushover analysis of various frames: the ultimate limit states for intact bridge substructure, functionality limit states for the intact bridge substructure, and the ultimate limit states for damaged bridge substructure. For the functionality limit state of the intact bridge substructure, a total lateral displacement at the bent cap equal to the free column height/50 (H/50) was recommended. This displacement considered as the point at the onset of plastic behavior.

Current NCDOT Design Practice

A review of the NCDOT's current driven pile bent design practices was undertaken. This review looked at current software, policies and specifications used to arrive at final design. A brief summary of the design procedure follows.

Using information from soil borings obtained at the proposed bridge sites and taking pile installation, pile availability, single pile capacity and possible pile displacements into account, one or more pile types and preliminary lengths are determined by the Geotechnical Unit. Next, one or more single pile lateral analyses are run using the LPILE program (Ensoft, 2004). Preliminary single-pile lateral analysis loads are assumed by the Geotechnical Unit for driven pile bents to be 2 kips in shear, the allowable axial load for the selected pile type, and no moment. In some cases, higher preliminary loads are given by the Structures Unit, which are then used instead of the assumptions. The moment and shear profiles, as well as the displaced shape of the pile are all considered, and, if necessary, the pile type is changed to meet a maximum displacement and loading criteria. NCDOT's practice places a lateral displacement limit of one inch at the pile top (at the cap level) which is used to determine pile acceptability under a given lateral load. There have been, however, times when lateral displacements greater than one inch were accepted after consultation with the designers in the Structures Unit.

The Geotechnical Unit then uses the LPILE results to determine a "point of fixity." (POF) This quantity is usually determined by few different methods. These include the point of maximum negative moment below the pile top, or the point of maximum negative displacement below the pile top. A sample POF definition is shown as Figure 12 in Chapter 3. The actual selection of the location of a point of fixity is determined by the geotechnical engineer's judgment.

Once the Structures Unit has the location of the point of fixity, the preliminary pile length, and the pile type, bridge design continues using frame analysis. Several loading configurations are applied and the Georgia Pier program is used for design.

Georgia Pier

Georgia Pier Version 4.2 is a linear elastic frame analysis program, developed by the Georgia Department of Transportation in 1993 and revised in 1994. The first Georgia Pier code was written in 1959 and upgraded for Load Factor Design (LFD) methodologies in 1980. The program can analyze up to 12 piles and was designed to analyze only reinforced concrete frames, so steel piles are currently accounted for using sections with equivalent properties.. The bent cap can have up to 36 gravity loads applied to the cap (Georgia Department of Transportation, 1994).

In Georgia Pier, wind loads are generated internally using AASHTO wind pressures and entered superstructure geometry parameters. Live loads and traction loads are calculated using the North Carolina Bridge Design Software (NCBDS) add-on. This subroutine calculates live loads at the bearing points according to AASHTO.

Georgia Pier in Design

Once the load cases and the geometry of the pile and bent cap sections have been entered in Georgia Pier, the minimum number and size of the longitudinal reinforcement bars must be selected for the bent cap. Similarly, a minimum size for the shear reinforcement must be entered.

Georgia Pier's output includes moments and shears along the bent cap, as well as the reinforcement needed to satisfy the moment demand or minimum specifications. If the minimum longitudinal reinforcement is not adequate, Georgia Pier calculates the required number of bars, which were sized in the input section. Similarly, Georgia Pier determines the required spacing of the stirrups for shear reinforcement based on the size entered in the input. Under service load conditions, Georgia Pier also checks stresses in the reinforcement for fatigue limits and crack width in the concrete according to AASHTO equations.

It should be noted that the Georgia Pier output results for piles are based on unfactored loads and are sorted by load cases that yields three maxima: maximum axial force, maximum transverse shear and maximum longitudinal shear. For all three maxima, the corresponding moments and shears for the corresponding load case causing the particular maxima are summarized. Based on these results, geotechnical design of the piles proceeds.

Observed Limitations of Georgia Pier

Over the course of reviewing the Georgia Pier inputs and results, a few limitations of the Georgia Pier design approach were observed. Some limit the breadth of design options. Others affect the results of the analysis.

General

1. Georgia Pier does not calculate displacements. Vertical, longitudinal or transverse displacements are not considered as a part of the analysis. In the geotechnical analyses, a single pile lateral displacement and sometimes a single pile vertical settlement are considered and reported to the Structures Unit if their magnitude is above approximately one inch.

Piles

2. Using LPILE with preliminary loads does not adequately model the behavior under lateral loads actually applied. The point of fixity chosen based on the preliminary load level should give similar results when placed in a frame only if the loads actually applied to the frame are similar. Another iteration using actual applied loads could optimize the design.

3. Pile data input into Georgia Pier assume the piles are vertical. According to engineers at the NCDOT, while there is some apparent capability for entering pile

batters, the feature gives erroneous results or no results at all. The working assumption is the batter will lower transverse displacement (i.e. results based on vertically oriented piles will be conservative).

4. For H-Pile sections, the NCDOT occasionally rotates the pile's strong axis such that different piles provide additional stiffness in either the transverse or longitudinal direction. For example, as shown in the plan view in Figure 1, a pile bent may have its two end piles installed on a batter with the strong axes in the transverse direction, while the interior piles have their strong axes in the longitudinal direction to resist traction forces and wind loads. Georgia Pier cannot model different piles each with its correct strong axis orientation.

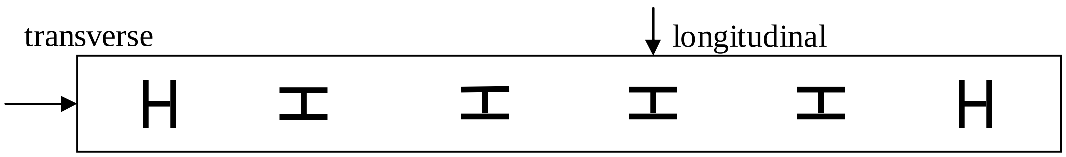


Figure 2. Plan View: H-Pile Bent with Rotated Pile Axes

5. For concrete pile sections, Georgia Pier requires that the concrete in the bent cap has the same elastic modulus as the concrete in the piles. Since the piles are typically precast and prestressed while the bent cap is typically cast in place, this is not the case. To correct for the different elastic modulus, the Georgia Pier analyst runs the program with an equivalent pile section. This section uses a moment of inertia, increased or reduced, such that the product of the bent cap elastic modulus and the actual pile's gross moment of inertia is equal to the product of the pile's elastic modulus and the equivalent moment of inertia.

6. Georgia Pier does not model soil, and therefore does not capture group effects, which tend to reduce stiffness compared to behavior of a laterally loaded single pile. Similarly, nonlinear lateral stiffness effects modeled in LPILE is theoretically captured using the point of fixity. The point of fixity should, based on current Georgia Pier outputs, allow for accurate modeling of moment or shears in the pile. Since displacements are not calculated by Georgia Pier, modeling the same lateral or axial stiffness as the lateral pile analyses (conducted by Geotechnical engineers) is not performed.

7. The k-values input in Georgia Pier are taken from AASHTO tables meant for steel columns in frames. These nominally determine the buckling behavior of an

unbraced column, and are currently used to magnify moments due to second order effects and to calculate a structural axial capacity of the pile. For a nonlinear analysis which takes into account second order moment effects as part of the algorithm, this k-factor would be redundant.

Analytical Limitations

8. The Georgia Pier program uses the method of moment distribution to obtain moments, shear, and axial forces. Like the Hardy Cross (1932) method of moment distribution, Georgia Pier uses an iterative method of approximations to find moments and rotations in each joint. Even though the joints are allowed to rotate, the model implemented in Georgia Pier does not consider any translation of the joints due to elastic shortening of the piles, differential settlements of the substructure, non-symmetric geometry or asymmetric loading conditions, and any other condition that may cause a vertical displacement of the joints. The results of these calculations will likely differ from those that would be experienced by the structure in the field under service conditions.

   In order to verify that Georgia Pier calculates moments and stresses based on Hardy Cross's method of moment distribution, a pile bent was solved manually using one cycle of moment distribution. This analysis was performed to obtain moments, shear, and axial loads. The pile bent was analyzed under a live load case without considering displacements in the pile-bent connections. When translation of joints was not considered, the results obtained manually were very similar to those obtained from Georgia Pier. This issue was verified further by comparing the Georgia Pier output to another model analyzed by the finite element program SAP2000, in which the joint displacements at the top of the pile were also restrained, the results were very consistent. Comparing, however, results from these models to the "real" behavior of the structure represented by a fully nonlinear SAP2000 model, yielded significant differences in results.

Once the analytical methods used in Georgia Pier and their limitations were identified, the verification software packages could be investigated. Software packages were chosen that included nonlinear lateral soil models and nonlinear section analysis options.

In summary, assumptions in current practice were established and shortcomings of the "preferred" design tool, namely Georgia Pier, were established. Limitations of the current practice were also identified.

CHAPTER 3: 3-D NONLINEAR MODELS

The main objective of the 3D- modeling was to determine if the designs from the existing 2-D linear frame model (Georgia Pier) could be improved. Accordingly, 3-D finite element models were developed. Such a model would have to be able to take into account the soil surrounding the piles and any pile group action. Similarly, such a model should be capable of accommodating nonlinear models for the piles and concrete cap. A number of programs were considered, including MultiPier, ANSYS, SAP and others.

Analysis Software

MultiPier

MultiPier (Bridge Software Institute, 2004) was selected to be the primary modeling tool for performing these analyses. MultiPier was considered most promising because it (1) had an interactive pile bent software wizard built in, (2) automatically modeled the soil resistance (lateral and axial, single and group) using methods representing the current state of geotechnical engineering practice, (3) allowed typical linear or nonlinear pile and bent cap models to be saved and utilized, and (4) had the option of modeling bridges with multiple bents connected by idealized superstructure elements.

MultiPier requires the input of soil parameters, pile type and length, bent cap dimensions and geometry, and AASHTO or LRFD load combinations. The program includes a wind load generator for wind loading conditions. The load combinations entered in this particular study were service or construction load combinations provided by the NCDOT, but the program's load input procedure is flexible enough to analyze any mix of construction, service or ultimate loading. Output can be displayed graphically, as a text file or as an XML file.

SAP

As a check to the results of MultiPier, SAP2000, version 8.2.5 (Computers and Structures, Inc., 2003) was used. SAP is a static and dynamic finite element analysis software package for structures. Using SAP2000, models for each bent were created, both as Point of Fixity-type frame analyses and with full length piles with nonlinear springs along the length to model pile-soil responses.

Practical Limitations of Both Programs

Both MultiPier and SAP can be used as analysis tools but both lack robust section design modules. As analysis tools, a bridge that has been preliminarily designed is inputted into the programs to find displacements, forces, moments, and other design quantities. These output results can be considered: against limit states, or against other design procedures.

In order to be utilized as a design tool, both programs would require additional pre and post processing. In this case, both MultiPier and SAP require some level of preliminary design work. Preliminary pile types, pile lengths, loads, bent cap geometry and bent cap reinforcement must be entered into the model. Wind loads can be generated in 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 loads would have to be calculated using a preprocessor.

After a preliminary design is entered, an initial analysis is run. The results from either program will include resultant forces and moments in the piles and bent cap, displaced shapes of the structure and its components and, in MultiPier only, demand capacity ratios for any section modeled as a nonlinear system. The moments, forces and displacements in the pile or bent cap can be used to determine whether the ultimate capacity of a particular member is adequate (by a factor of safety or using load-resistance factors) for a predicted amount of movement. Once these values are calculated, the bent cap rebar can be verified or determined using equations or methods outside the programs. Alternatively, a post processor can be developed to capture the output files from these analysis programs, aggregate the loads and moments into critical envelopes for all load cases, and perform section design.

In addition, a postprocessor would particularly be needed to check the bent cap reinforcement design. It should be noted that the Georgia Pier program already performs this calculation: the minimum specified reinforcement is entered, along with other acceptable bar diameters. If the minimum is acceptable, nothing changes, but if the minimum is insufficient, Georgia Pier calculates the new required reinforcement, or shape for the section.

Soil Modeling

To model the interaction between soil and pile, the built in soil models in MultiPier were used. In early discussions with the NCDOT, it was determined that the soil models for lateral analysis included in MultiPier were the same models used by the department for LPILE analysis, as both utilize the P-y approach. Based on the prior success of these lateral models, the built in models were used in this analysis.

Lateral (P-y) Model

Lateral pile load-deformation analysis is performed by modeling the pile as a series of elastic masses and the soil as a series of nonlinear springs. The spring stiffness of each spring is

The lateral pile-soil interaction models available in MultiPier are summarized in Table 2. All are summarized briefly in the MultiPier manual (BSI, 2000) but are discussed in their original literature in detail.

Table 2. MultiPier P-y curve models

Material	Developer	Used in this project
Sand	Reese, Cox and Koop (1974)	Yes
Sand	O'Neill (1984a)	
Soft Clay, Below Water Table	Matlock (1970)	Yes
Stiff Clay, Below Water Table	Reese *et al.* (1975)	Yes
Stiff Clay, Above Water Table	Reese and Welch (1975)	
Clay	O'Neill (1984b)	

Once the model parameters were input and the curves developed in MultiPier, approximations of those models were transferred to SAP. Because SAP 8.2.5 has no built-in P-y soil models, those generated by and displayed in MultiPier's soil input section were used. In MultiPier, once the soil parameters are input, the user can view plots and tabular P-y curves to be used in the analysis as shown in Figure 2. To transfer to SAP, 20 data points calculated by MultiPier were copied as load-deformation definitions for non-linear springs representing soil surrounding each pile.

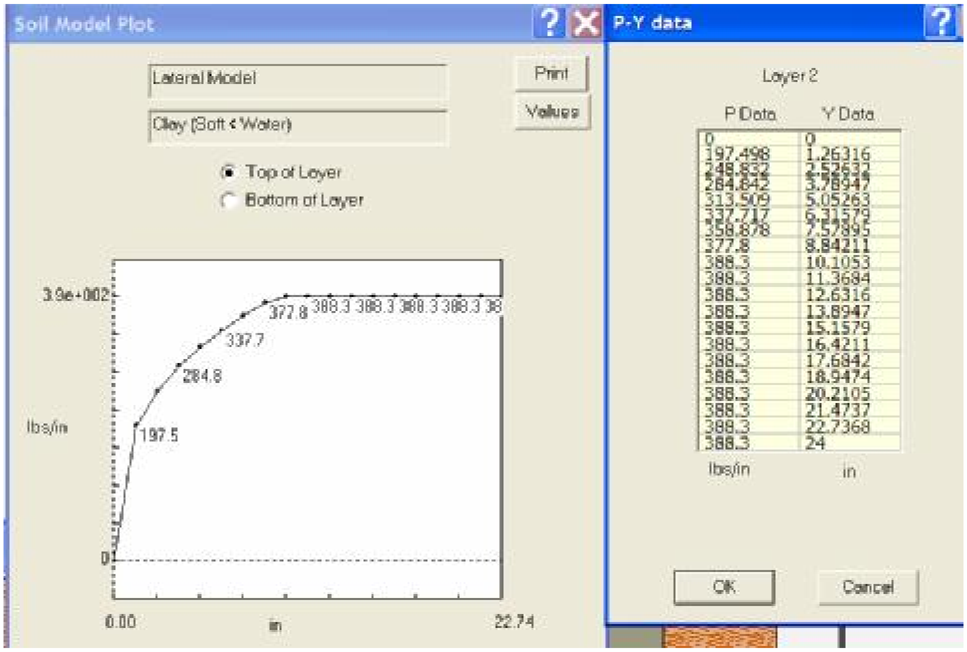


Figure 2. MultiPier In-Program Soil Model Display

MultiPier presents tabular results for both the top and the bottom of a selected soil layer. Presumably, MultiPier assigns a lateral soil spring load-displacement curve for every node. Any interpolation between the model parameters at the top and bottom of the layer in MultiPier is linear. In SAP, however, the P-y curves from the top and the bottom of the layer were used as starting points for linear interpolation between the two depths. While not perfect, this resulted in values as close as could be expected to what was actually used in MultiPier. This ultimately led to some of the differences in transverse and longitudinal displacement observed, particularly in the lateral analyses. Specifically, two major differences arose:

1. MultiPier presented tabular values in twenty equal displacement increments, as shown in Figure 2. This incremental spacing usually meant insufficient resolution for the initial, highly nonlinear portion of the P-y curve. For small displacements, therefore, the lateral soil reaction force in SAP was mostly lower than that actually predicted by the MultiPier analysis. This lower force meant the system was less stiff, which meant SAP produced higher displacements. This will become apparent in the results section of Chapter 4.

2. 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 given the ultimate capacities at the top and bottom of the layer increases linearly with

depth. Both ultimate lateral capacity curves are shown in Figure 3. SAP's linear increase would lead to lower maximum forces compared to MultiPier, and therefore underpredicting the stiffness and over predicting the displacements.

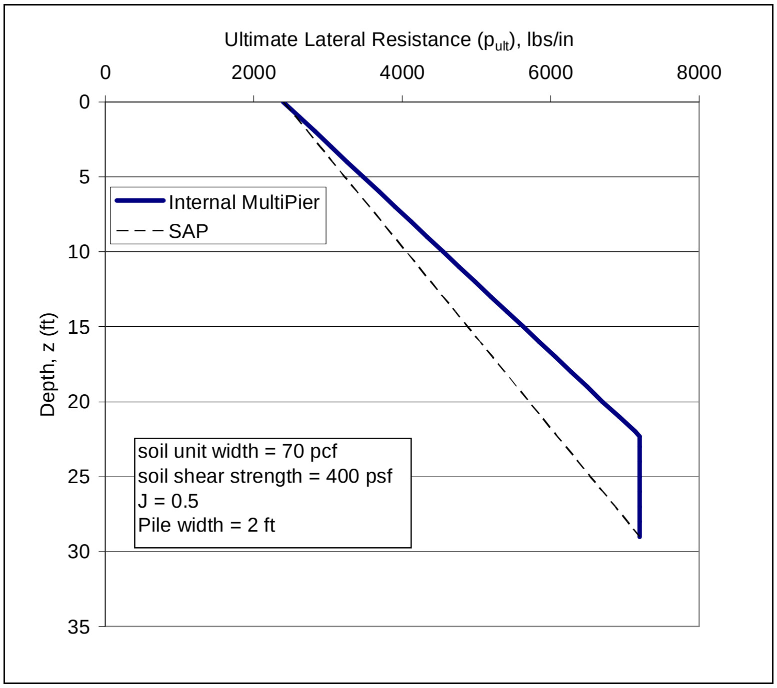


Figure 3. Variation between SAP and MultiPier lateral maximum capacity.

Axial (t-z), Skin Friction

MultiPier uses an asymptotic shaft resistance model. The mathematical formulation used is shown in Equation 3.1 (McVay et al., 1989), while a typical plot of unit shaft resistance versus displacement is shown if Figure 4. The initial stiffness of the curve is dictated by the low strain shear modulus, while the asymptotic behavior is determined by the equation's final bracketed term, which is also a function of shear strength. The shear modulus is then effectively reduced by the bracketed term as the applied shear stress increases, which is captured by the "B" term as follows (McVay et al., 1989).

$$z = \frac{r_o t_o}{G_i}\left[\ln\frac{(r_m - B)}{(r_o - B)} + \frac{B(r_m - r_o)}{(r_m - B)(r_o - B)}\right] \tag{3.1}$$

where:

z = axial (vertical) displacement

τ_o = shear stress being transferred from pile to soil

τ_f = ultimate shear stress between pile and soil

G_i = soil initial (low strain) shear modulus

r_o = radius of the pile

r_m = radius where vertical soil displacements are negligible. See Eq. 3.3, (Randolph 1978)

$$B = \frac{r_o t_o}{t_f} \tag{3.2}$$

$$r_m = L(1-n)\frac{G_{center}}{G_{tip}} \tag{3.3}$$

L = pile length

ν = soil Poisson's ratio

G = Soil shear modulus at pile center or pile tip

Similar to the P-y curves shown in Figure 2, the t-z curves at the top and bottom of each soil layer can be viewed in MultiPier. A table of data can be displayed and these were copied and transferred to SAP. Once in the SAP preprocessor file, the values are multiplied by the pile perimeter and the length of the element used at each point in the model. In both SAP and 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 MultiPier consider equally spaced displacements. As a result, the resolution of the early nonlinear portion of the curve is often coarse. Thus the SAP shaft resistance model is often less stiff than that used in MultiPier.

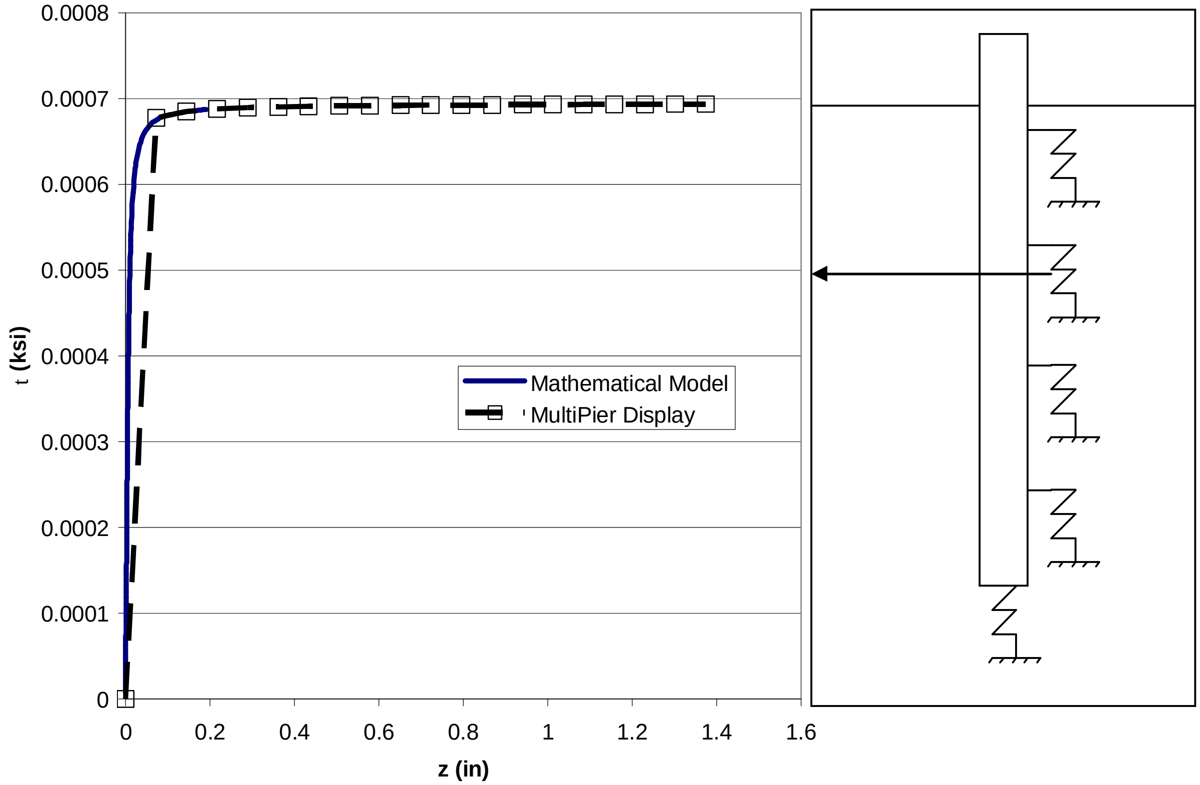


Figure 4. Example shear stress vs. displacement curve, 18 inch pipe pile in sand, G_i =3.5 ksi

Axial (q-z), End Bearing

Initially, the built-in MultiPier end bearing model was used in both SAP and MultiPier to model the pile-soil interaction. The pile toe model was developed by McVay, et al. (1989). The mathematical expression is shown in Equation 3.4.

$$z = \frac{Q_b(1-n)}{4r_oG_i\left[1-\frac{Q_b}{Q_f}\right]^2} \qquad (3.4)$$

Where:

Q_b = End bearing force applied to toe

Q_f = Ultimate End bearing force

Other variables are the same as shaft resistance model.

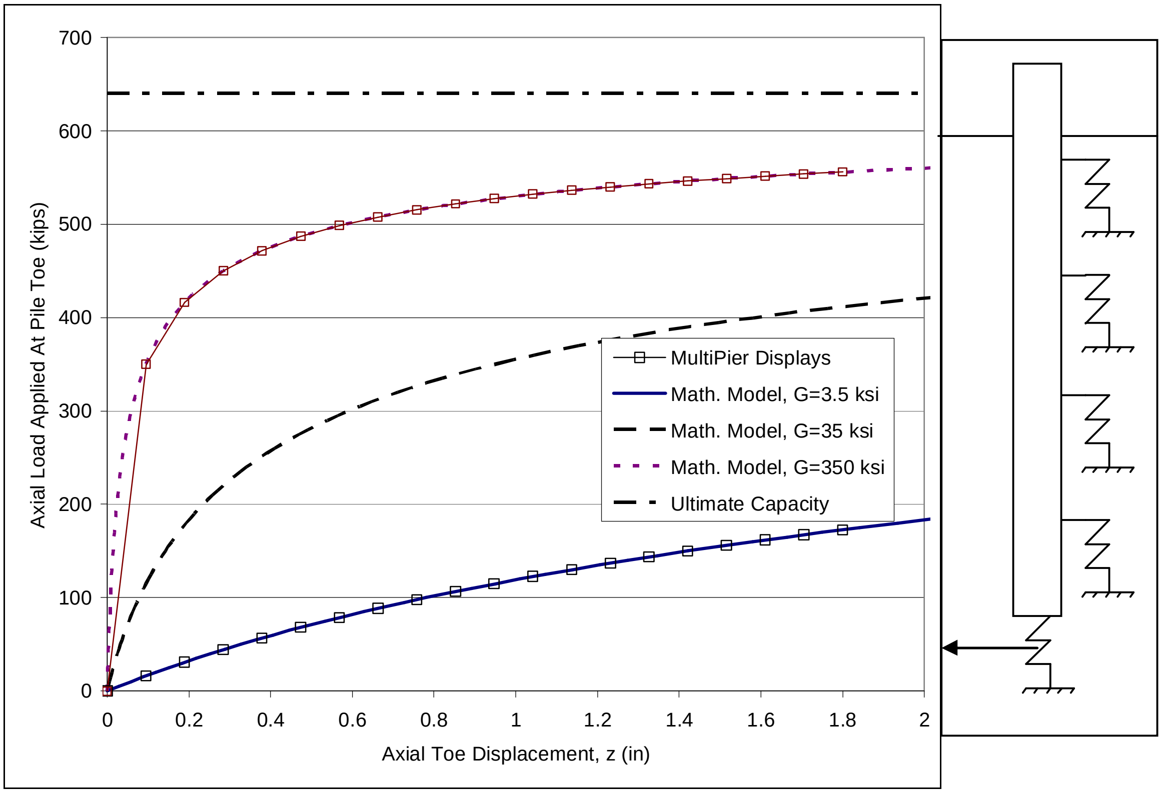


Figure 5. MultiPier load-displacement curves at the toe, various shear moduli, G

As can be seen in Figure 5, the MultiPier toe response is very sensitive to the value of initial shear modulus input. In Figure 5, an ultimate toe resistance of 640 kips was input. In most load test interpretation methods, *total* pile capacity is sometimes defined as the capacity mobilized at displacements equivalent to 1 and 10% of the pile diameter or width. Most static toe resistance prediction models such as Vesic's (1977), Kulhawy et al.'s (1983), Meyerhof's (1976) or others tend to give ultimate resistances that are mobilized at relatively low displacements.

Using standard static resistance models in the MultiPier model, however, tend to create a situation in which reaching the capacity requires very large displacements. In Figure 5, to obtain 90% of the 640 kip ultimate capacity on an 18 inch diameter pipe with an initial shear modulus of 3.5 ksi (a value typical of a soft clay), the model predicts the pile must move more than 300 inches. At an initial shear modulus of 35 ksi (a value typical for a dense sand), the pile must move 30 inches, or nearly 3 feet. Even at a very high soil shear modulus of 350 ksi (shown here for numeric purposes), the pile must move three inches to mobilize 90% of this example's ultimate capacity.

If the MultiPier toe model is to be used, it would have to be looked at very carefully from a geotechnical standpoint. The toe resistance model would have to be considered in light of the NCDOT's experience with partially weathered rock and in soils on the coastal plain. If the model tends to overpredict displacement, either the ultimate capacity parameter will have to be increased (ultimately derived from the soil's shear strength), or the shear modulus would have to be increased to model the actual field behavior reasonably.

While considering initial results from the bridge models using the MultiPier toe resistance model, it was noted that high vertical displacements were occurring both globally and differentially. The differential displacements between two piles within a single bent generated large moments in the bent cap, particularly under the predominantly live load cases of Group IA. In discussing the results with NCDOT engineers, the toe model's response was apparently not stiff enough. The vertical differential settlement is not likely to occur in the field since the live load is applied relatively quickly and equally among the bridge lanes (versus modeling it as a factored static load) and the piles are generally driven to relatively high blow counts.

Accordingly, it was decided to "fix" the pile toe. In this case, a model was adopted to allow a small amount of movement at the toe. An ultimate capacity of 1000 kips was arbitrarily chosen for these piles. The load-displacement curve would increase linearly until a displacement of 0.1 inches was reached, then plastic failure at 1000 kips was modeled as shown in Figure 6. It should be mentioned that, for these piles, 1000 kips of total factored load was not applied.

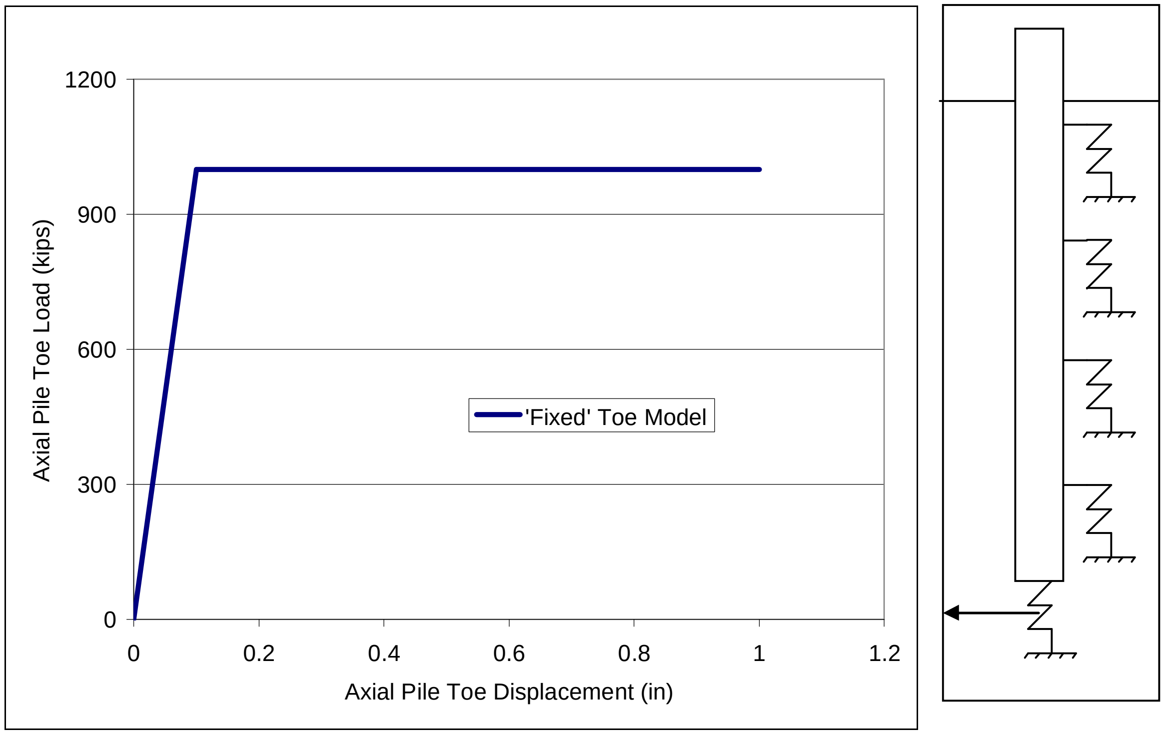


Figure 6. "Fixed" Pile Toe Displacement q-z Model used in lieu of more detailed PWR model

Future research could be directed at finding a more appropriate toe model for the weathered rock and coastal plain regions. This could be deduced from static load tests in the NCDOT archives or from high strain dynamic tests.

Model Input Parameters: Initial Shear Modulus

The MultiPier manual (BSI, 2000) suggests estimating an elastic modulus from N-value (Eq. 3.5 to 3.7) , then calculating a shear modulus based on the Poisson's Ratio (Eq 3.8). McVay et al. (1989) also used equation 3.9, a correlation between shear modulus and CPT tip resistance after Robertson and Campanella (1984) .

$$E(ksf) = 10N_{60} \text{ (silty or clayey sands)} \tag{3.5}$$

$$E(ksf) = 20N_{60} \text{ (Normally consolidated sands)} \tag{3.6}$$

$$E(ksf) = 30N_{60} \text{ (Over consolidated sands)} \tag{3.7}$$

$$G = \frac{E}{2(1+n)} \tag{3.8}$$

$$G(ksi) = 0.125 q_c (tsf) \tag{3.9}$$

In these equations, E is the elastic modulus at a strain level likely developed during SPT testing, N_{60} is the SPT N-value, corrected for energy transfer, G is the shear modulus, ν is the soil's Poisson's ratio, and q_c is the CPT tip resistance.

Figure 7 shows estimated shear modulus from equations 3.5 to 3.9. This figure uses a Poisson's ratio of 0.3 and Robertson et al.'s (1983) correlation of SPT N-value to cone penetration resistance, Figure 7 is presented with two x-axes, one for N-value and the other for qc. In general, for sands, the ratio of qc (tsf) to N-value is between 4 and 6. Because much of the residual soils in North Carolina have significant fine content, a ratio of 4 was used. This figure shows shear modulus for sands to be estimated at between 1 and 10 ksi.

As shown in the sample toe resistance curve in Figure 5, shear modulus values of around 3.5 ksi lead to a very soft toe response. The values obtained based on equations 3.5 to 3.9 do not seem to be low strain shear modulus values. Since the shear modulus determines the initial stiffness of the toe response, other approaches for evaluating low strain shear modulus were examined. Figures 8 and 9 show shear modulus measurements from resonant column and torsional shear tests which are small strain approaches.

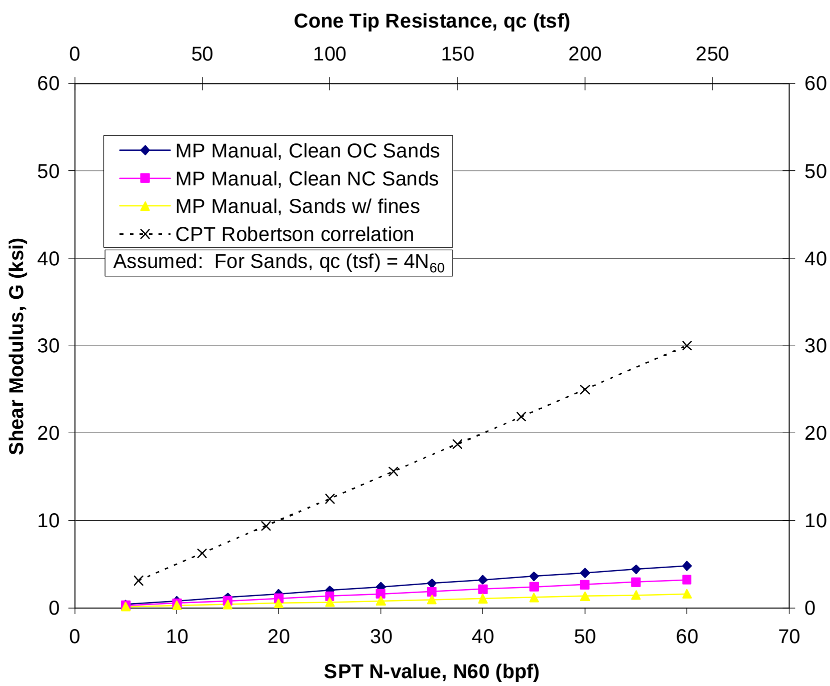


Figure 7. Shear modulus to SPT N-value correlations

Figure 8 shows low strain shear modulus of sands from Harden and Black's 1968 work. Figure 9 shows low strain shear modulus values for North Carolina residual soils after

Borden and Shao's 1995 study. Note the difference in x-axes between Figure 7 (*in situ* tests) and Figures 8 and 9 (effective confining pressure). Also note the large difference in y-axis scales. These Gmax values are more suitable to typical soils in the Piedmont and Coastal regions; values for partially weathered rock and the rock of the mountain region are unavailable. For the bridge bent t-z analyses presented in Chapter 4, the initial shear modulus values presented in Figures 8 and 9 were used, although it appears equation 3.9 would be reasonable, as well.

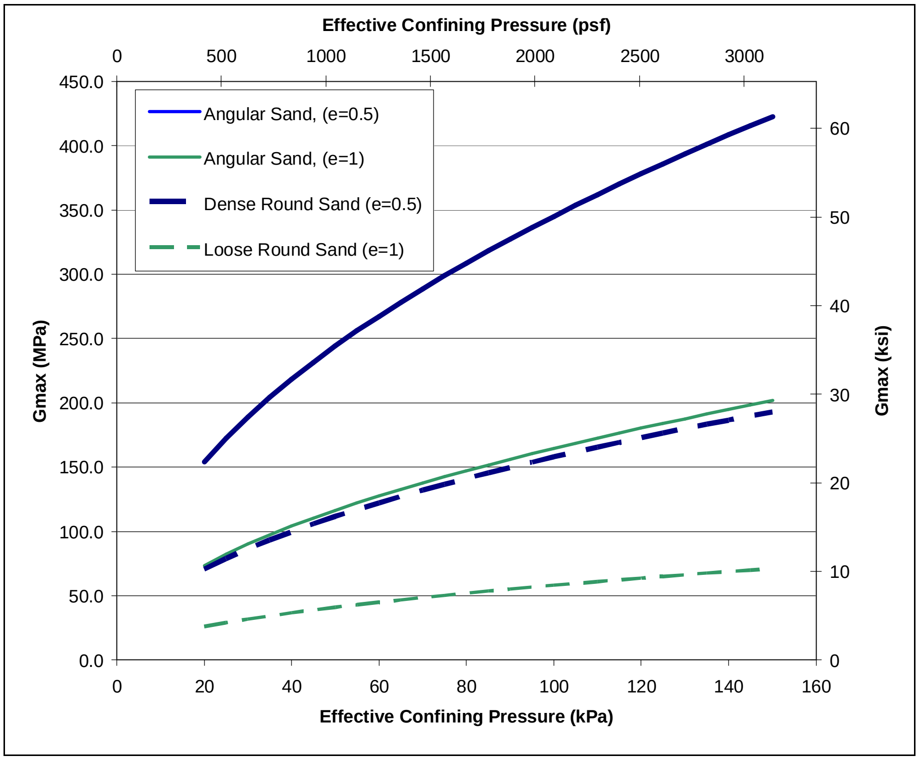


Figure 8. Initial (Low Strain) Shear Modulus for Sands (Harden and Black, 1968)

Subgrade Modulus

Calculation of the P-y curves requires the estimation of subgrade moduli, k, for the top and bottom of each soil layer. The analysis approach presented herein used the k-values presented in the MultiPier manual (BSI, 2000). These figures are similar to those recommended in the L-PILE (Ensoft, 2004) program's manual and are reproduced as Figure 10.

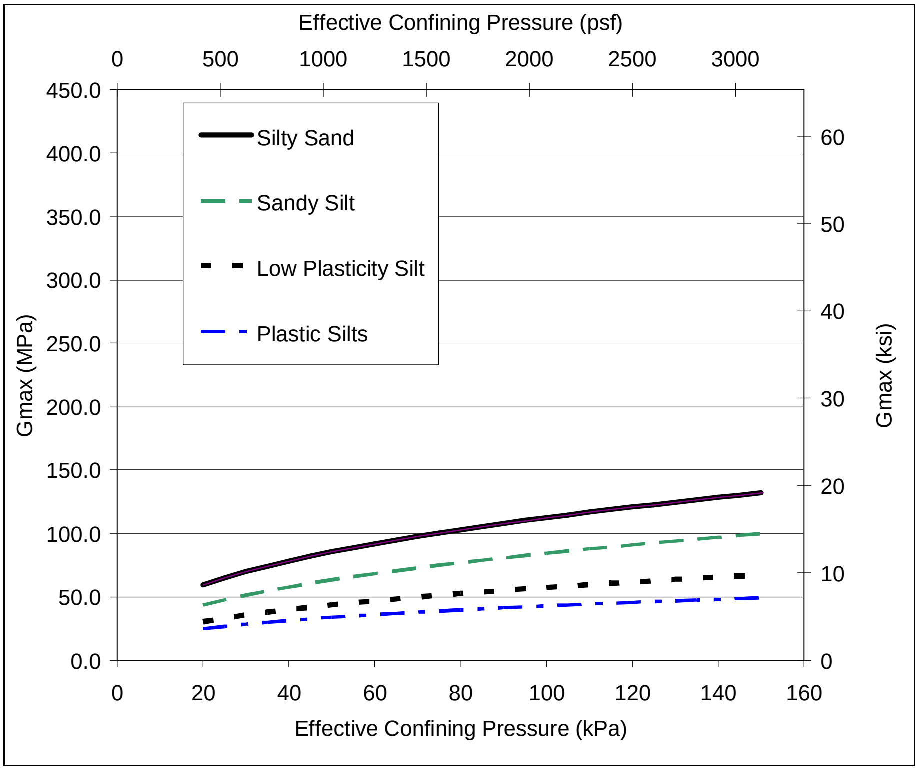


Figure 9. Initial (Low Strain) Shear Modulus for North Carolina Residual Soils (Borden and Shao, 1995)

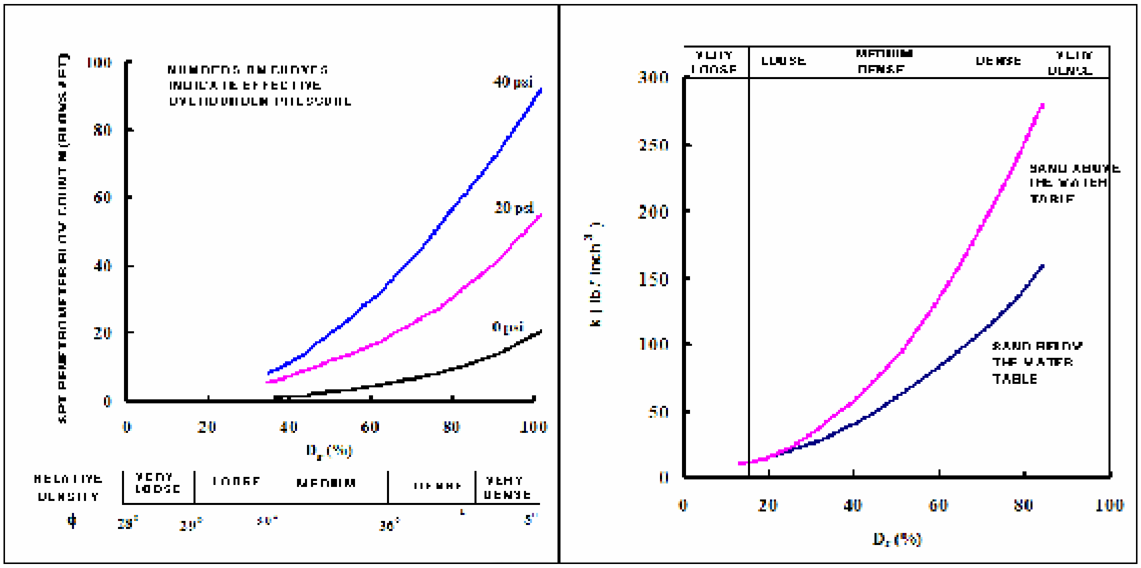


Figure 10. Determination of subgrade modulus from SPT N-value (MultiPier Manual)

Nonlinear Response of Structural Elements

The nonlinear analyses as described in this chapter require a detailed estimation of the material and section properties of the structural and foundation elements. MultiPier presents several features for modeling structural elements, such as the use of the Mander model for confined concrete in circular RC columns. This model accounts for the increase in compressive strength and ultimate strain of concrete as an effect of the confinement provided by the transverse reinforcement. Another feature is the use of a fiber model for steel, composite and RC sections. In MultiPier, all components of the section response are automatically integrated for each step of loading, and therefore the user does not need to input section response data (i.e. axial-moment-curvature response).

In the case of SAP2000, the nonlinear modeling of the structural elements, requires the input of axial-moment-curvature response for all sections in the model. A moment-curvature response curve, such as that presented in Figure 11, depicts the moment developed in a section when a certain value of curvature has been induced or vice versa. In this project moment curvature response curves were computed for all pile and cap beam sections, and these curves were entered in SAP2000 for the nonlinear analysis.

The moment curvature analysis were performed mainly using King program (Priestley and Park, 1986), which was developed by M.J.N. Priestley and was supervised by R. Park at the University of Canterbury. Like MultiPier, this program uses the Mander model to determine the stress-strain response of confined concrete (Mander et al., 1988). The program considers the confining effect of transverse steel, the nonlinear behavior of the concrete and reinforcement steel, the spacing of longitudinal steel, the axial load effect in the flexural capacity, and other factors.

Additional verifications were performed using Response 2000 (Bentz, 2001), which uses the Modified Compression Field Theory (Vecchio and Collins, 1986), and bi-linear approximations calculated by hand. All three methods characterized the reinforced concrete sections by an estimate of an initial secant cracked moment of inertia, and a yield point. The cracked moment of inertia was used to linearly model the cap-beam section in MultiPier .

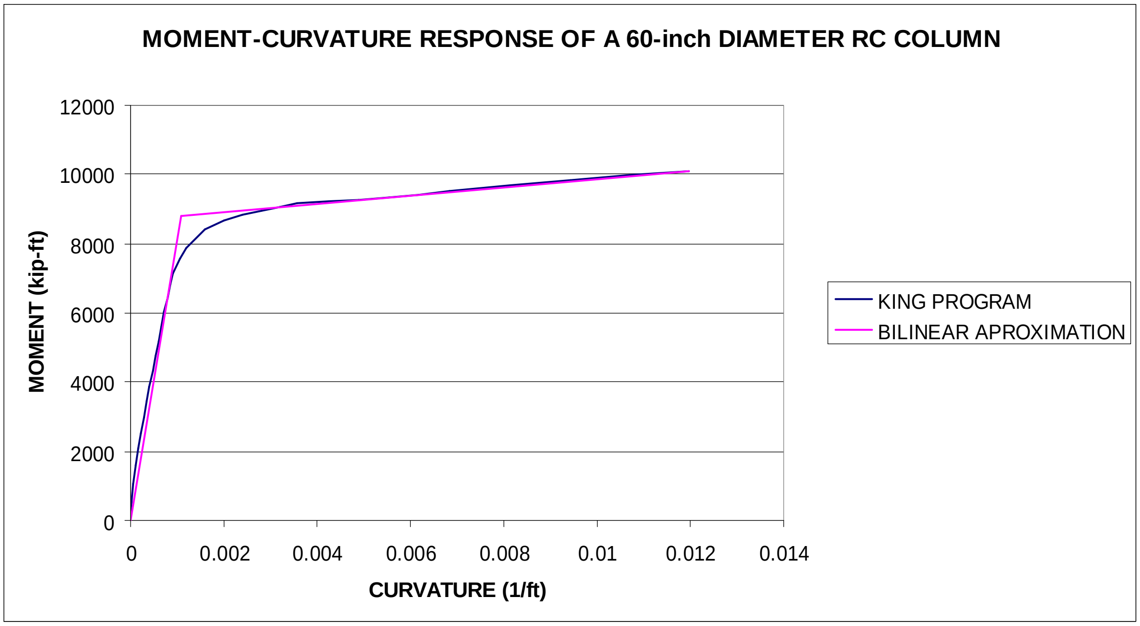


Figure 11. Moment Curvature Analysis on a Concrete Column

MultiPier Verification

Before modeling each of the four NCDOT bridges submitted, a series of basic, smaller models were built to determine how MultiPier worked and how its results compared with other programs.

Soil-pile interaction: Nonlinear lateral single pile analysis

Data for the four bridges analyzed included documentation from LPILE (Ensoft, 2004). LPILE is a software package that performs lateral analysis on single piles using a P-y curve approach. As such, it is similar to MultiPier's treatment of lateral pile deflection under loading. The pile parameters and soil parameters included in the LPILE output were entered into MultiPier's single pile analysis subroutine.

The LPILE analysis modeled a single HP14x73 that was 54 feet long. The loads applied to the pile top consisted of a 2 kip lateral load applied parallel to the H-Pile's strong axis and 120 kips of axial dead load. The soil profile was modeled as two layers of sands overlying a very stiff clay layer (partially weathered rock). The first sand layer was 26 feet thick and was modeled with a unit weight of 122.9 lb/ft^3, a friction angle of 28 degrees and a subgrade modulus of 70 lb/in^3. The second sand layer was 22 feet thick with the same unit weight and friction angle as the upper sand layer. The subgrade modulus in the deeper sand layer was 100 lb/in^3. The clay layer in which the pile terminates was modeled with a unit weight of 122.9 lb/ft^3, an undrained shear strength of 6.5 ksf, a subgrade modulus of 1300 lb/in3, and a major strain of 0.004 at 50% of the maximum applied stress.

The results from both the initial L-Pile run and MultiPier runs with both 18 and 50 nodes are shown in Figures 12 and 13. The deflected shapes for all three analyses are quite similar, with a slightly smaller pile top deflection predicted for the 50 node MultiPier case. The magnitude of the maximum moment is similar for all three runs, as well. The results of the comparison between LPILE and MultiPier presented here are similar to the results from an earlier study presented in the MultiPier manual (BSI, 2000).

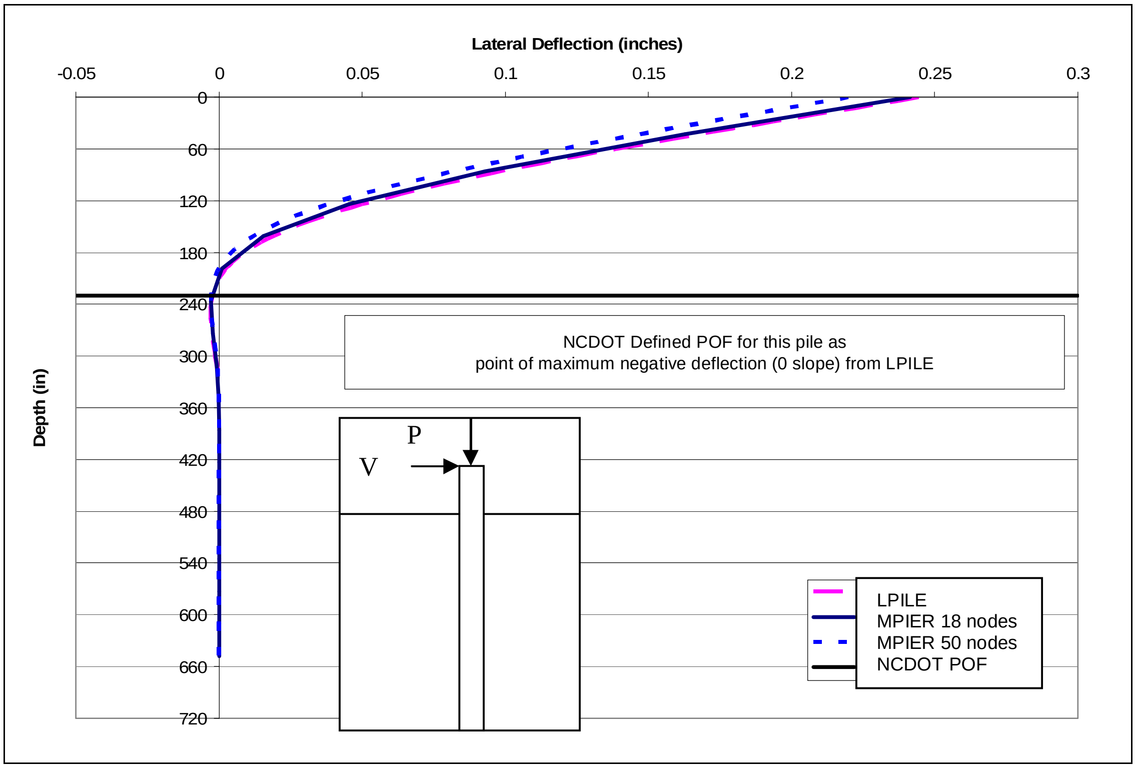


Figure 12. LPILE and MultiPier Single Pile Deflected Shape

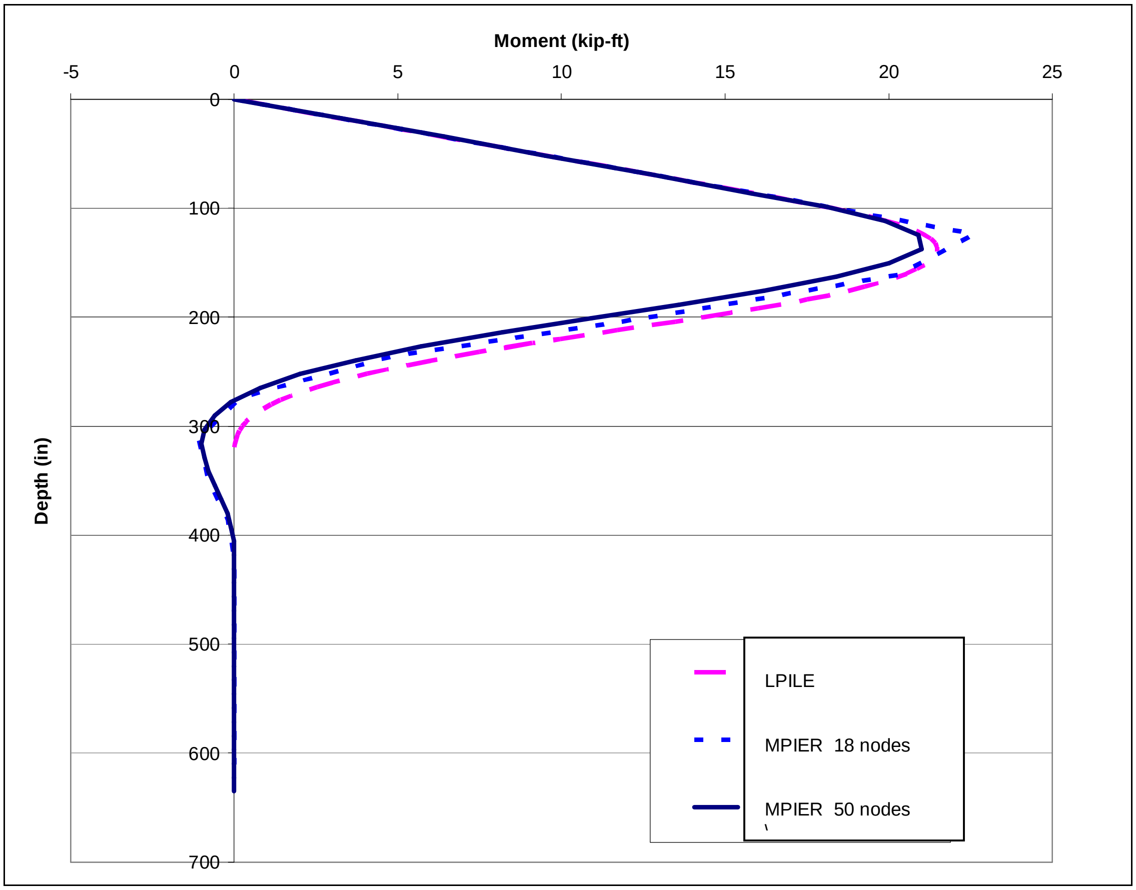


Figure 13. LPILE vs MultiPier Single Pile Moments

Nonlinear structural modeling: Pushover analysis of a reinforced concrete cantilever column

A test case was run to verify the capabilities of MultiPier for predicting nonlinear response of reinforced concrete elements. The results from MultiPier were compared to the results of a similar analysis performed in SAP2000.

The column used in this verification was a 60-inch diameter, 30 ft long reinforced concrete column. The compressive strength of the concrete was assumed to be 5.2 ksi, the longitudinal reinforcement was 25 # 14 bars with an assumed yield strength of 68.9 ksi. The transverse steel was a #5 spiral at 3.5 inches. The cover over the main bars was four inches. To perform the pushover analysis, an incremental lateral load was applied at the top of the column while a 1000 kip axial load was kept constant. The bottom of the pile was fixed.

Figure 14 shows the results of the analyses performed using both MultiPier and SAP. To run this analysis in SAP, the moment curvature response of the section had to be input. Figure 11 shows the moment curvature response as given by the King program (Priestley and Park, 1986) and also shows a bilinear approximation of that response. The bilinear approximation was used in SAP to model the column.

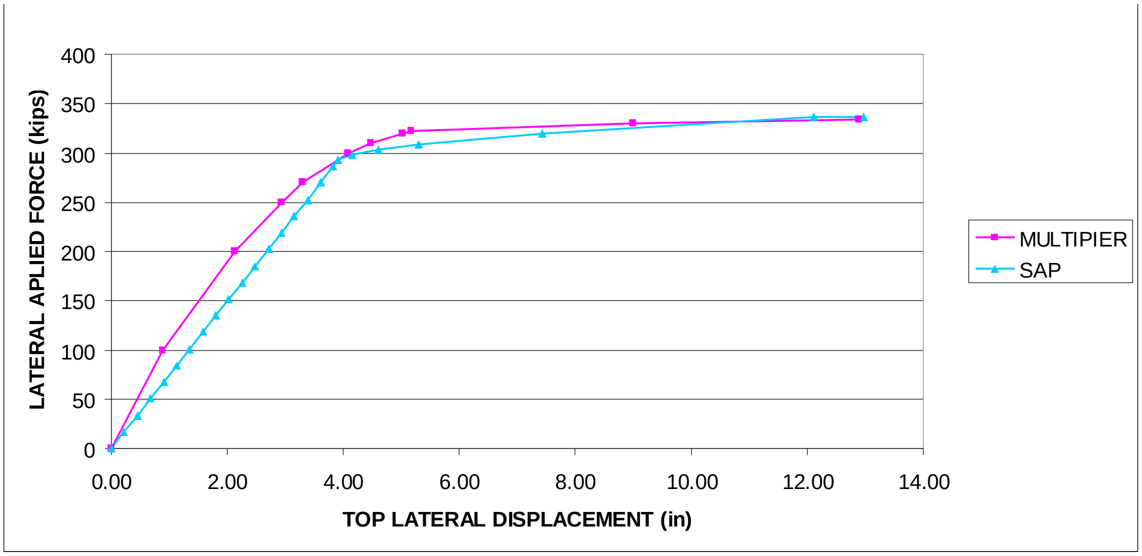


Figure 14. Concrete Column Pushover Analysis Results—MultiPier and SAP

The results of the pushover analysis using both programs are similar as shown in Figure 14. The response obtained from SAP is nearly bi-linear, because a bilinear approximation for the moment curvature response of the section was used as an input. The results from the MultiPier analysis are more representative in the sense that all the stages of the nonlinear response, including initial cracking of the section, are captured.

AASHTO Load Cases and Output Values for Design

The AASHTO Loading Groups must be defined in the nonlinear models. In a single MultiPier file, the allowed number of load cases includes one dead load case, up to five wind load cases and up to nine live load, longitudinal forces, impact forces, or other cases. MultiPier also includes a built-in wind load generator. SAP has no such limitation on the number of load cases, but the load cases are not built-in.

In this project, AASHTO Groups I, IA, II and III were considered. It was reported by NCDOT engineers that the Group IA case (1.3*Dead Load + 2.86* Live Load +

2.86*Impact Load) is not typically considered in design. They are, however, included in these analyses as a basis for the bent designs to represent worst case scenarios.

The MultiPier results were prepared for comparison to SAP results with the following steps:

(1) Find the summary at the end of the MultiPier text file.
(2) Examine the maximum pile top 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 piles.
(3) For each maxima, identify the AASHTO load case and (if applicable) pile number associated with the maxima.
(4) Using the SAP graphical output results, the same load case from MultiPier is checked for the desired maxima and displacement, moment or force envelope for the desired node.
(5) Compare the output results from SAP and MultiPier.

For both the MultiPier and SAP verification analysis, the concrete bent cap models were entered as linear elastic. The required parameters for a linearly defined bent cap model are primarily section area and moments of inertia. Because the amount of reinforcement for the already designed bridges was known, a cracked moment of inertia could be estimated using the section analysis techniques described earlier. Once this cracked moment of inertia was entered into both SAP and MultiPier, the maximum moments could be identified and the required amount of rebar calculated.

In practice, the method above should work relatively well during the design phase. Once a basic bent cap shape is selected, the gross moment of inertia can be calculated and the cracked moment of inertia can be roughly estimated using ACI design recommendations (2005). ACI recommends reducing the gross moment of inertia by 50% to arrive at an initial cracked moment of inertia estimate. Using this initial value, moments can be calculated, and rebar can be sized. The longitudinal reinforcement should follow LFD AASHTO specifications.

If the above approach is used, the design could be optimized in one of two ways. If SAP is used, a section analysis program would be employed to more properly calculate the cracked moment of inertia. The model could then be re-run with the new cracked moment of inertia, the moments recalculated, and the rebar design checked. If MultiPier is used, the bent cap concrete and rebar could be entered as a nonlinear section, and the moments checked that way.

For all bridges, the calculated rebar may not fulfill the requirements of the 2003 NCDOT Bridge Design Manual. The minimum rebar requirements are summarized in Table 3. For this project, all rebar calculations are performed and presented, followed by the minimum rebar required by the Design Manual for comparison.

Table 3. NCDOT 2003 Bridge Design Manual Longitudinal Reinforcement Requirements

Bent Cap Width	Minimum Main Rebar, U.S. size (metric)
Less than or equal to 3 ft (910mm)	4 #9 (#29)
Between 3 and 4 ft (910 to 1220 mm)	5 #9 (#29)
Between 4 and 5 ft (1220 to 1520 mm)	6 #9 (#29)
Between 5 and 5 ft, 8 in (1520 to 1730 mm)	7 #9 (#29)

The maximum pile top displacements are compared to allowable displacements identified for the structure. In current DOT practice, this displacement limit is usually set at one inch, unless the structural engineer determines more displacement is acceptable. Chapter 6 will include proposed approaches to determine limit state developed for this project, in addition to those discussed in the Literature Review in Chapter 2. In many cases, however, results from MultiPier and SAP will usually not converge if the lateral or vertical displacements are predicted to be higher than 5 or 6 inches.

The axial pile forces and moments are used in MultiPier, along with the nonlinear pile section properties to calculate a demand capacity ratio (DCR). A DCR of 1.0 or greater implies failure, either in compression or buckling. A DCR of less than 1 implies the section is able to carry the factored loads applied by the range of AASHTO loading cases entered for the analysis.

Bridge Models

Once the initial verifications were completed, the four bridge case studies submitted by the NCDOT were modeled. Selected drawings of each bridge from NCDOT design records are included for reference in Appendix A. The next chapter describes the nonlinear MultiPier and SAP analyses for the four bridges in detail, as well as optimization of the pile foundation in each case.

CHAPTER 4: 3-D NONLINEAR MODELS AND, RESULTS

The plans and design documents from four bridges were submitted to the project team to analyze with detailed 3-D models, namely SAP2000 and MultiPier. The bridges were selected to capture a variety of pile types, superstructure types, and soil conditions. One bridge is relatively long; the other three are relatively short. A brief summary of each bridge follows.

In these analyses, the transverse direction is parallel to the bridge's cap beam, as shown in Figure 1. The longitudinal direction is perpendicular to both the bridge's cap beam and the axis of the drive piles. The axial direction is perpendicular to the cap beam and parallel to the axis of the vertical driven piles.

Robeson County Bridge, Project B-3507

This bridge spans the Lumber River on State Route 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. A sketch of the interior pile bent is included in Appendix A.

General Information

Designed: 2002

Spans: 2 (30 and 40 feet)

Interior Bents: 1

- **Pile Type:** Eight HP14x73; 55 ft long; end piles battered 1:8
- **Free Pile Length:** 5 ft (without scour); 8 ft (with scour)
- **Bent Cap:** 36 inch wide by 30 inch deep Class A concrete beam
- **Cap Reinforcement:** Five #9 bars (top) and four #10 bars (bottom)

End Bents: 2

- **Pile Type:** Eight HP12x53; 55 ft long; Four brace piles battered 1:4
- **Free Pile Length:** None
- **Bent Cap:** 33 inch wide by 30 inch deep (minimum) Class A concrete beam with wing walls

Superstructure: Fifteen 3 ft by 1.75 ft prestressed concrete cored slab units

Interior Bent Super/Substructure Connection: Two rows of 15 elastomeric bearing pads, ¾ in. thick, Type II (Expansion Joints)

End Bent Super/Substructure Connection: One row of 15 elastomeric bearing pads, ¾ in. thick, Type I (Fixed Joints)

Section Analysis

The section analysis for the 36 inch wide by 30 inch deep concrete bent cap is summarized in Figure 15. The moment curvature analysis results are shown in Figure 15 along with hand calculations and results from both Response 2000 (Bentz, 2001) and the King Program (Priestley and Park, 1986). All results agree quite nicely, and the ultimate capacity and cracked moment of inertia given the as-built dimensions and reinforcement are indicated in Figure 15. The cracked moment of inertia was used in the linear cap models for both SAP and MultiPier.

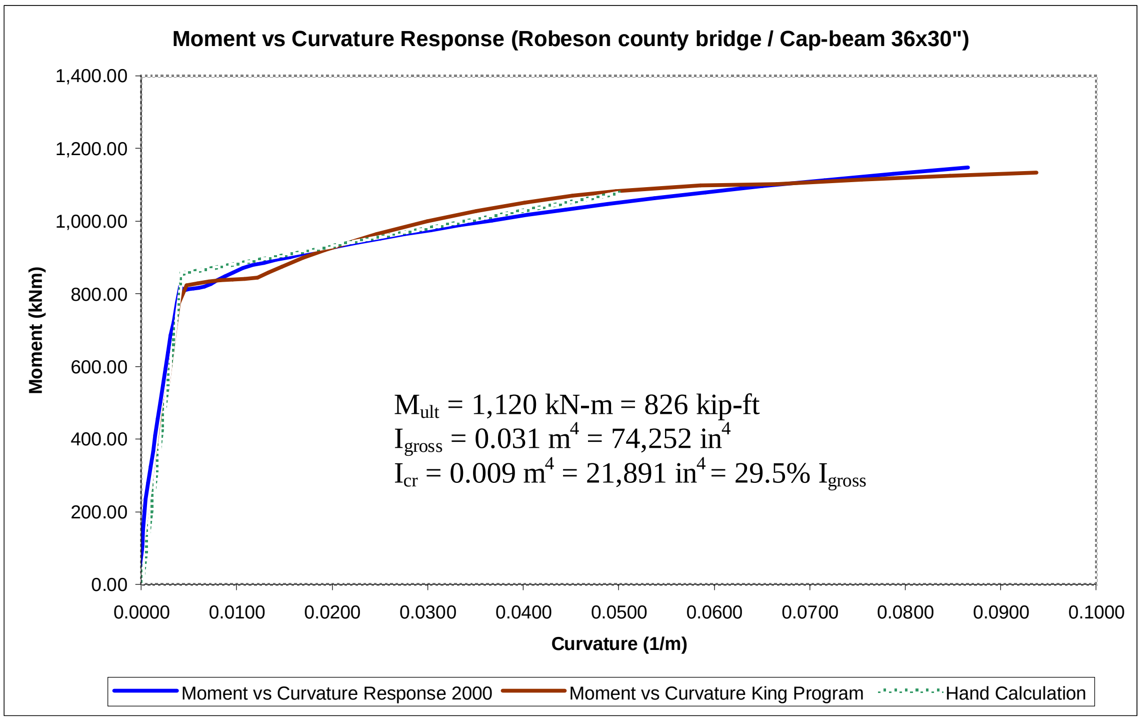


Figure 15. Robeson County Moment Curvature Analysis

Geotechnical Summary

For analysis of this bridge case study, the interior bent was modeled. The soil profile at this bent was summarized in boring B1-B. The soil boring shows the ground water level to be at the surface. From up to 3 feet below the ground surface, very loose silty sand was reportedly encountered. 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 feet, with N values averaging 21 blows per foot. Fine to coarse sand was reported from 12 to 22 feet, with N values ranging from 4 to 15 blows per foot.

From 22 feet to 43 feet, the coarse sands of the Black Creek Formation were encountered. N-values were in the teens through this layer, except for one sample at 29.7 feet that dropped to 2 blows per foot. Very stiff silty clay material was encountered from 43 to 53 feet, with very high N-values of 62 and 70 blows per foot. The boring terminated 66 feet below ground surface, with dense fine to coarse sands.

As designed, the piles were to terminate in the 62 to 70 blow count silty clay material. Based on discussions with the NCDOT engineers, it was likely assumed in design that this material is partially weathered rock (PWR). The end bearing material was therefore rather stiff. Based on the project data, pile capacity at the shaft and at the toe was estimated for the preliminary analysis. Because the PWR was considered to be quite stiff, a second toe model was suggested that somewhat arbitrarily limited displacement to 0.1 inches at 1000 kips. This second model allowed a small amount of axial pile displacement to occur.

Lateral group analysis considered the spacing between the piles, which for this bridge was 72 inches, or slightly greater than five times the 14 inch width of the pile (5D). From the MultiPier manual (BSI, 2000), the leading pile's P-y multiplier was 1, the adjacent pile's multiplier was 0.85 and all other piles' multipliers were 0.7. For the 5D spacing, axial group capacity was considered to be unaffected.

Analysis Results—SAP and MultiPier

Models of the bridge pier were created in both MultiPier and SAP. Figures 16 and 17 show the two models, respectively. The input files can be found in the Electronic Appendix. The results are summarized in Table 4. The pile toe was effectively fixed.

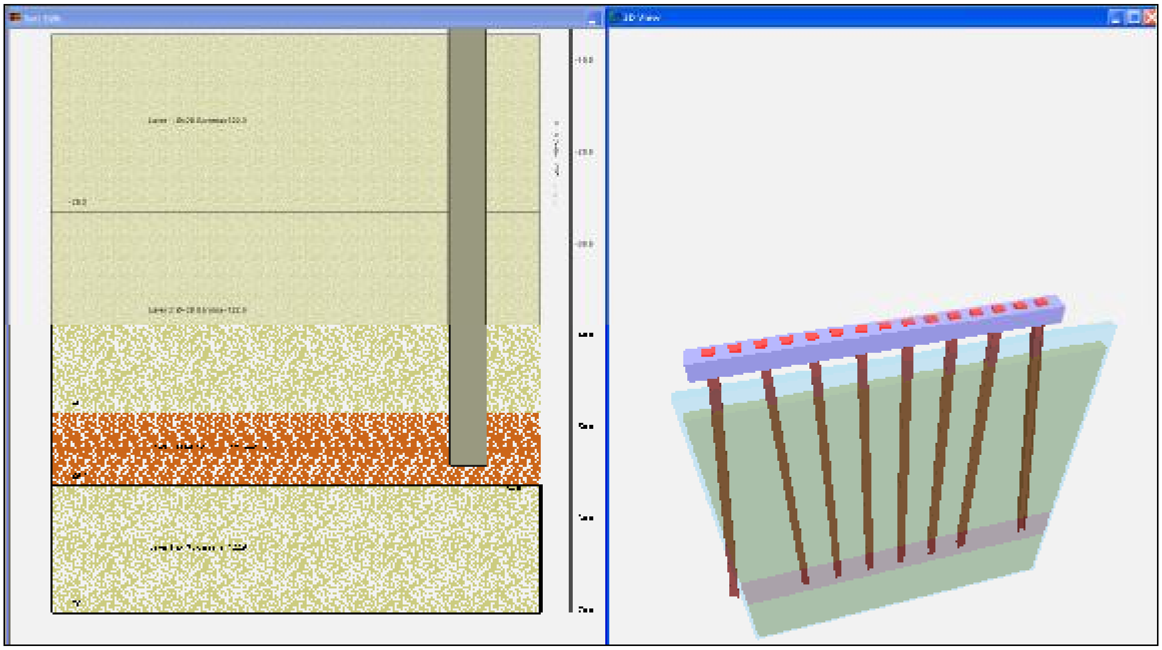


Figure 16. MultiPier Model--Robeson County Bridge

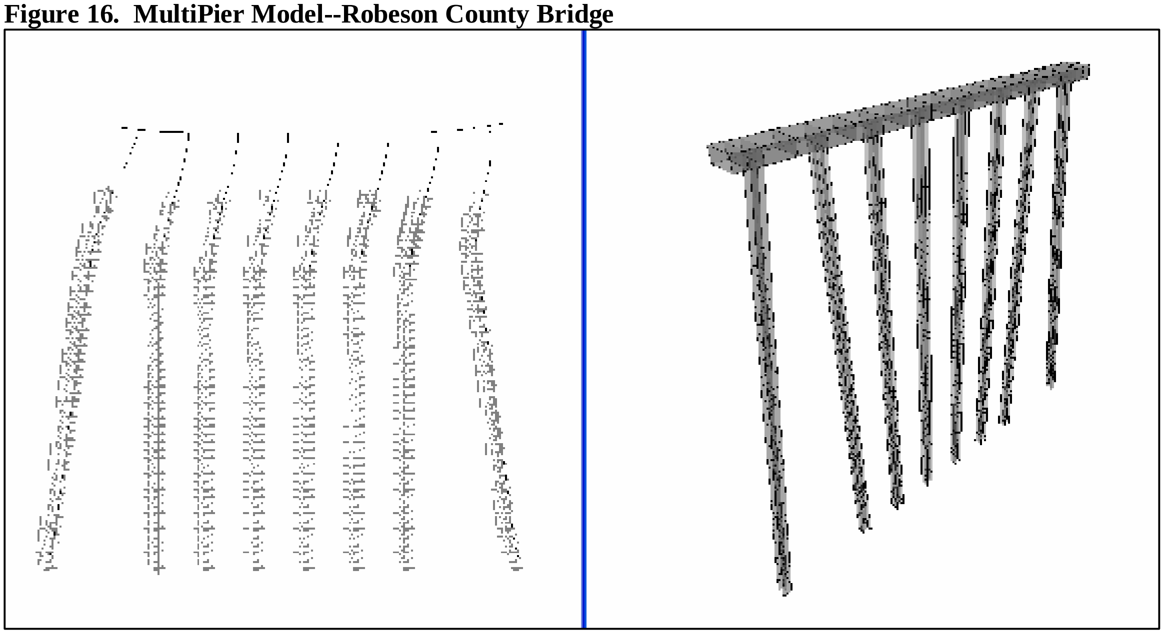


Figure 17. SAP Model--Robeson County Bridge, Deformed and Unloaded Shapes

Table 4. Robeson County Nonlinear Analysis Results

Model	Axial Pile Top Disp (in)	Transverse Pile Top Disp (in)	Longitudinal Pile Top Disp (in)	Maximum Moment in Bent Cap (kip-ft)
SAP	0.20	0.09	0.33	172
MultiPier	0.21	0.10	0.28	182
AASHTO Group	1A-LL4 Pile 3	2-WS1 Pile 1	2-WS5 Pile 6	1A-LL8

Analysis Results—Optimization

After the MultiPier models were verified by SAP, MultiPier was used to optimize the design by reducing the number of piles and the size of the cap beam. The bent was constructed from eight HP14x73 piles with a 36 inch by 30 inch cap beam. The cap beam's reinforcement was five #9 bars on top and four #10 bars on the bottom. According to NCDOT specifications, the minimum number of bars for a 36 inch wide cap beam is four #9 bars. The total length of the cap beam was assumed to be dictated by the required geometry of the deck and roadway and was unchanged for this optimization.

First, the eight HP14x73 piles were replaced by HP12x53 and HP10x42 sections. In reducing the size of the piles, it was assumed that the pile toe resistance model would stay the same--the toe would still be approximately fixed. Since we are assuming the toe is bearing in a partially weathered rock-like material, this assumption may be acceptable.

A summary of the results is shown in Table 5. The installation and material cost was taken from the NCDOT 2004 Bid Averages, which averages all bids from contractors for a given year. The HP10x42 piles were noted as not available (N/A) because these piles have not been included in the bid averages since 2001. In that year, the smaller HP10x42 were actually more expensive than the HP12x53. Due to the lack of cost data, and because the displacement in the longitudinal direction of the HP10x42 would only get larger than 1.08 inches if the number of piles were reduced, it was decided the HP12x53 piles would be used to further investigate potential reductions in the number of piles.

Table 5. Robeson County Bridge Alternative Pile Configurations

	HP14x73	HP12x53	HP10X42
Demand/ Capacity Ratio (Piles)	0.24	0.33	0.35
Displacement, transverse (pile top, in)	0.10	0.19	0.33
Displacement, longitudinal (pile top, in)	0.28	0.54	1.08
Displacement, axial (pile top, in)	0.21	0.28	0.35
Bent Cap Maximum Moment (kip-ft)	182	202	221
Cost Per Linear Foot (2004)	$45.50	$34.75	N/A

Table 6 shows the results from reducing the number of HP12x53 piles. Since the number of piles or piles' size was reduced, the bent cap width could also be reduced from 36 inches to 33 inches while still maintaining the required concrete cover. This reduction, however, may not fulfill the required set-backs from the cap edge for the elastomeric bearing pads. The rebars noted in Table 6 are minimum required to satisfy the expected moment, but do not provide for minimum values based on AASHTO specifications.

Table 6. Robeson County Bridge, Reducing Number of HP12x53 Piles

HP12x53	8 Piles	7 Piles	6 Piles	5 Piles
Demand/ Capacity Ratio (Piles)	0.33	0.38	0.44	0.55
Transverse Displacement (pile top, in)	0.19	0.24	0.33	0.31
Longitudinal Displacement (pile top, in)	0.54	0.68	0.81	1.29
Axial Displacement (pile top, in)	0.28	0.34	0.39	0.50
Bent Cap Moment Max (kip-ft)	202	214	298	346
Minimum Rebar (Top/Bottom)	4 #5	4#6	4#6	4#7
	4 #6	4#7	4#8	4#8
Minimum Rebar (NCDOT Specs)	4#9 (top/bottom)	4#9 (top/bottom)	4#9 (top/bottom)	4#9 (top/bottom)

Table 6 shows that the number of piles can be reduced from eight HP14x73 to as low as five HP12x53 without exceeding the overall structural capacity of the piles or exceeding the maximum moment in the bent cap. Indeed, even with only five piles, the bent cap reinforcement could be reduced from #9 bars to #7 and #8 bars. Using the installation and material costs from the bid averages in Table 5, and assuming a 55 foot long pile, a foundation cost can be calculated. Since the bridge was constructed using eight HP14x73 piles, an estimated savings can be determined by subtracting the foundation cost of the "optimized" designs from the "as-built" design. Table 7 summarizes the estimates of these savings.

Table 7. Robeson County Bridge, Estimated Foundation Costs from 2004 NCDOT Bid Averages

Pier Layout	Total Pile Cost*	Cost Savings
Eight HP14x73	$20,020	As Built
Eight HP12x53	$15,300	$4,720
Seven HP12x53	$13,390	$6,630
Six HP12x53	$11,480	$8,540
Five HP12x53	$9,565	$10,455

Northampton County Bridge, Project B-3214

This bridge spans the CSX Railroad on US 301. It is a three span bridge. Each interior bent was constructed using pipe piles with a concrete cap; the abutments are H-pile supported with wing walls. A sketch of the interior pile bent is shown in Appendix A.

General Information

Designed: 2002

Spans: 3 (61, 120 and 49 feet)

Skew: 39^{o}-46'-14"

Interior Bents: 2

- **Pile Type:** Five 24-inch diameter closed end steel pipe piles, 0.5 inch thick wall; 60 ft long; end piles battered 1:8
- **Free Pile Length:** ~7 to 8 feet
- **Bent Cap:** 50 inch wide by 39 inch deep Class A concrete beam
- **Cap Reinforcement:** Six #9 bars (top and bottom)

End Bents: 2

- **Pile Type:** Seven HP12x53; 70 ft long; Four brace piles battered 1:4
- **Free Pile Length:** None
- **Bent Cap:** 30 inch wide by 30 inch deep (minimum) Class A concrete beam with wing walls

Superstructure: Four steel girders with cast-in-place concrete slab

Interior Bents Super/Substructure Connection: 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)

End Bent Super/Substructure Connection: One row of four elastomeric bearing pads (Type II, 2-7/16 inch thick)

Bearings: Bearings at the end bents are both fixed. Bearings at Bent 1 are expansion and fixed. Bearings at Bent 2 are both expansion.

Section Analysis

The section analysis for the 50 inch wide by 39 inch deep concrete bent cap described above is summarized in Figure 18along with a summary of hand calculations and results from the King Program (Priestley and Park, 1986). Both results are similar, and the ultimate capacity and cracked moment of inertia, given the as-built dimensions and reinforcement, are shown on the figure. The cracked moment of inertia was used in the linear cap models for both SAP and MultiPier.

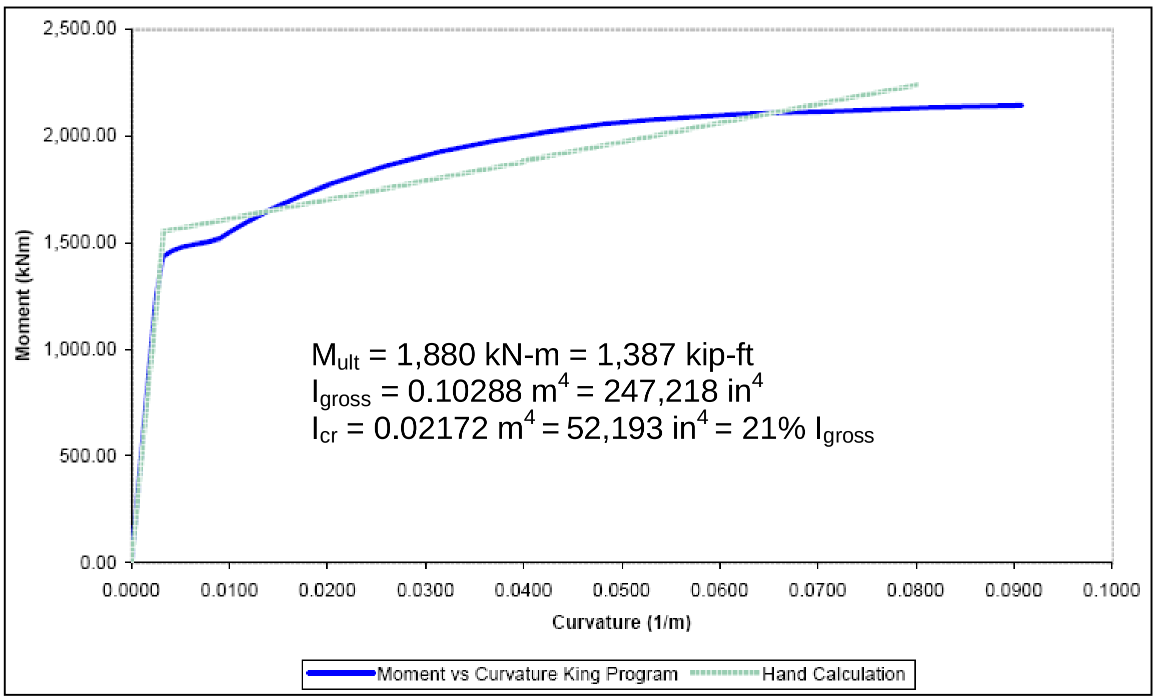


Figure 18. Northampton County Moment Curvature Analysis

Geotechnical Summary

For this analysis, one of the two interior bents was modeled. Two soil borings were performed by NCDOT for each of the two interior bents. They were labeled B1-A and

B1-B for Bent 1, and B2-A and B2-B for Bent 2. All four borings showed similar profiles. As such, Bent 1 was modeled in both SAP and MultiPier, and it was assumed the results for Bent 2 would then be similar.

The profiles can generally be described as composed of approximately 40 feet 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 eight feet below the ground surface. Because this bridge spans railroad tracks, there should be no reason to consider scour effects.

Boring B1-A first indicates a seven foot thick layer of clayey silty sand which has N-values greater than 10 blows per foot. An approximately 21 foot thick layer of sandy clay with N-values between 0 and 5 blows per foot was reportedly encountered next. From 28 to 37 feet, materials described as sandy silt were encountered. N-values of 3 to 8 blows per foot were reported for this layer.

From 37 to 52 feet, clayey silty fine sand was indicated. The N-values at the top of this layer ranged from 6 to 8 blows per foot, and increased to 25 and 27 blows per foot 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 foot, while 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 deeper than 62 feet ranged from clayey silty sand with N-values between 10 and 78 blows per foot 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.

Similar to the Robeson county bridge, the modeling of Northampton county bridge assumed the pile toe was fixed. In this case, however, the piles were end bearing in sand, not in partially weathered rock. In the absence of experience with both pile displacements under the pseudo-dynamic live loads and without confidence in the output of the toe models provided by MultiPier in North Carolina's soils, a fixed toe model was deemed appropriate, if not ideal.

Lateral pile group analysis considered the spacing between the piles, which for this bridge was 126 inches, slightly greater than five times the 24 inch width of the pile (5D).

From the MultiPier manual, the leading pile's P-y multiplier was 1, the adjacent pile's multiplier was 0.85 and all other piles' multipliers were 0.7. For the 5D spacing, axial group capacity was considered to be unaffected.

Analysis Results—SAP and MultiPier

The models for the Northampton bridge were developed in MultiPier and SAP as shown in Figures 19 and 20, respectively. Input files are included in the Electronic Appendix. Table 8 shows the maximum displacements at a pile top and maximum moment generated in the bent cap for this bridge. Both the MultiPier and SAP models yielded consistent pile responses under the applied load cases.

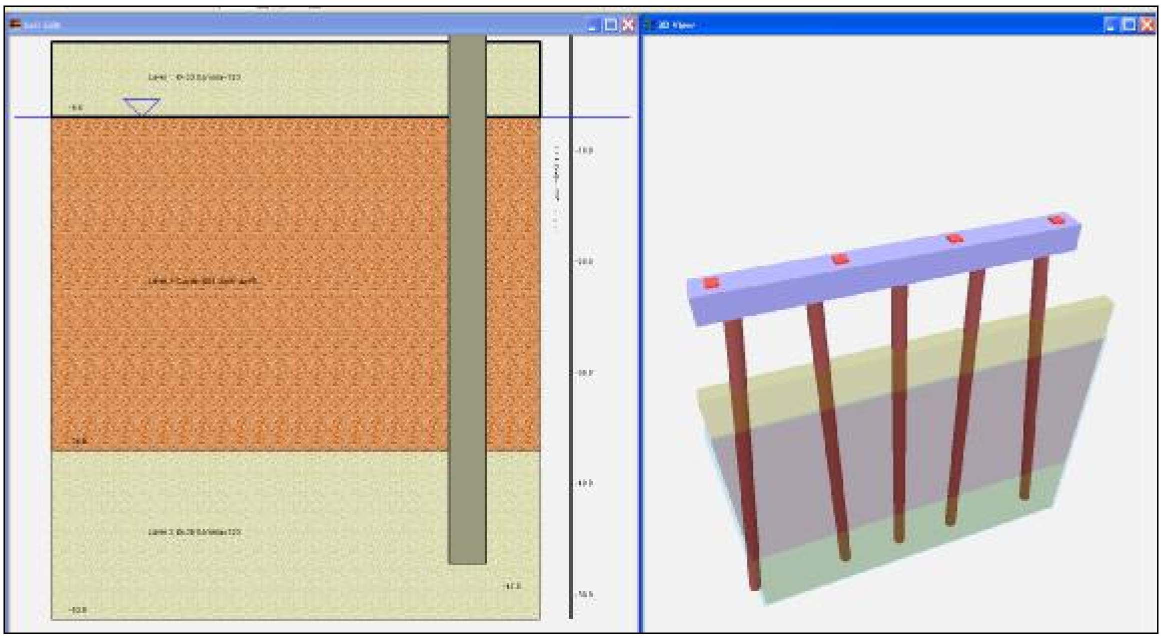


Figure 19 MultiPier Model--Northampton County Bridge

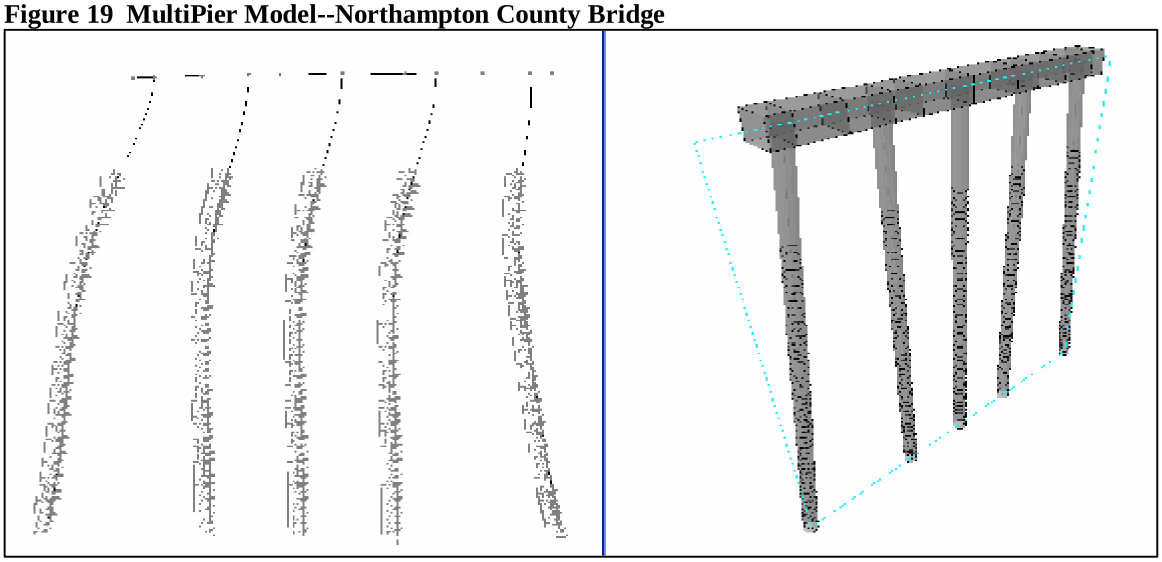


Figure 20. SAP Model—Northampton County Bridge, Deformed and Unloaded Shapes

Table 8. Northampton County Bridge, Nonlinear Analysis Results

Model	Axial Pile Top Disp (in)	Transverse Pile Top Disp (in)	Longitudinal Pile Top Disp (in)	Maximum Moment in Bent Cap (kip-ft)
SAP	0.23	0.26	0.89	733
MultiPier	0.25	0.23	0.72	638
AASHTO Group	1A-LL5 Pile 1	2-LL1 Pile 1	2-LL5 Pile 5	1A-LL1

Analysis Results—Optimization

Once the MultiPier result was verified, the model was then optimized by reducing the number or size of the piles while maintaining the same load cases. For this bridge, five 24 inch diameter closed end pipe piles were used. The walls were 0.5 inches thick, and the piles were 60 feet long. The bent cap used six #9 bars both on the top and bottom, and was 50 inches wide by 39 inches deep. Table 9 shows the results from reducing the number of piles.

Table 9. Northampton County Bridge, Reducing number or size of piles

	24" Pipes 5 piles	24" Pipes 4 piles	18" Pipes 5 piles	HP14x73 5 piles
Demand/Capacity Ratio (Piles)	0.37	0.46	0.49	0.46
Transverse Displacement (pile top, in)	0.23	0.29	0.45	0.94
Longitudinal Displacement (pile top, in)	0.72	0.94	1.75	4.11
Axial Displacement (pile top, in)	0.25	0.31	0.34	0.44
Bent Cap Moment Max (kip-ft)	638	164	660	682
Minimum Rebar (Top, Bottom)	5#8 5#9	5#5 5#4	5#7 5#9	5#7 5#9
Minimum Rebar (NCDOT Specs)	6#9	6#9	6#9	6#9
Pile Cost Per Linear Foot (2004)	$146.25	$146.25	$94.12 (est)	$45.50

As shown in Table 9, a relatively high longitudinal displacement is estimated for the 18 inch pipe piles and especially the HP14x73 piles. While displacements in the longitudinal direction are likely limited by the geometry of the bridge and relative freedom of movement of the expansion joints and connections to the superstructure, it seems more than four inch of longitudinal displacement for the HP14x73 piles is unlikely to meet design criteria.

The interesting result obtained by reducing the number of piles is the reduction in maximum bent cap moment from 638 to 164 kip-ft as the number of piles changed from five to four. This reduction in moment occurs because the bearing locations were changed from between two piles to directly over each pile, as shown in Figure 21. Because the loading point is no longer offset from the axis of the pile, the load can be transferred directly to the pile. As a result, the maximum moment due to the AASHTO load combinations are reduced, as shown in Figure 22. This reduction in maximum moment leads to reducing the required rebar from six #9 bars at the top and bottom to five #5 bars on top and five #4 bars on the bottom.

The estimated reduction in foundation costs is summarized in Table 10. The cost per linear foot of pile materials and installation are from the NCDOT 2004 Bid Averages for the HP14x73 and the 24 inch diameter pipe. The 18 inch diameter pipe was not available in the 2004 bid averages, but was in the 2003 bid averages. Based on the results from other steel piles, the 2003 average cost for 18 inch pipes was increased by 30% to approximate the 2004 cost of pipes.

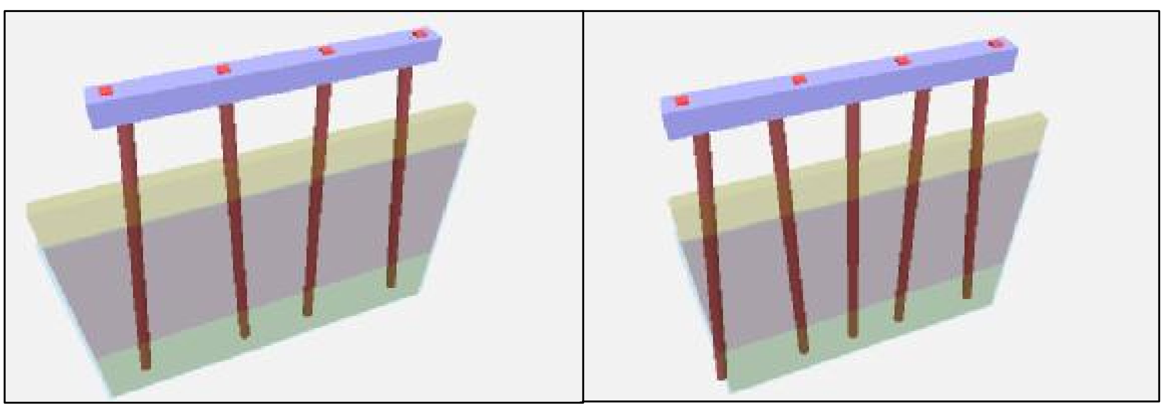


Figure 21. Northampton Bridge Models—Four and Five Pile Bent. Note location of bearing pads

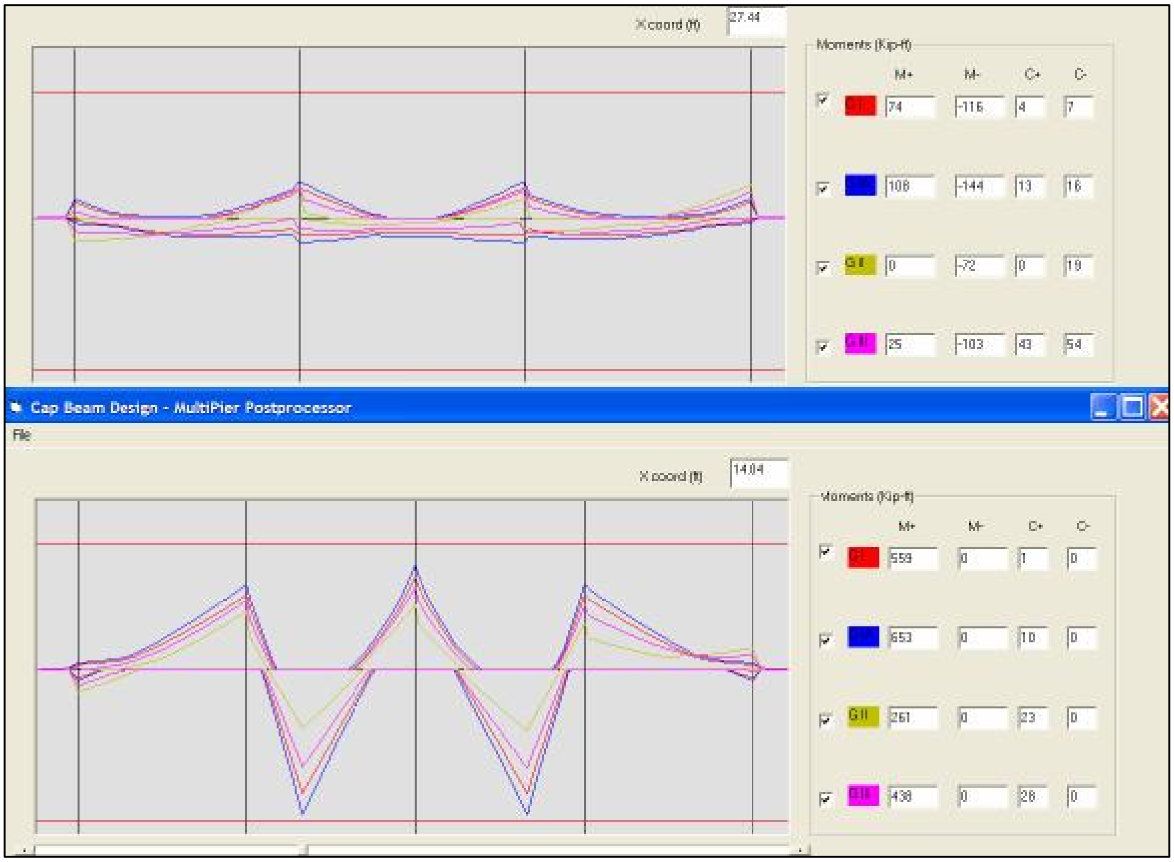


Figure 22. Northampton Bridge Cumulative Moment Envelopes--Four (top) and Five (bottom) Piles

Table 10. Northampton County Bridge, Foundation Cost Estimates

Layout	Total Pile Cost	Cost Savings
Five 24” Pipe	$43,875	As Built
Four 24” Pipe	$35,100	$8,775
Five 18” Pipe	$28,236 (est)	$15,639
Five HP14x73	$13,653	$30,222

As can be seen both from Table 9 and Table 10, the 24 inch pipe piles were relatively expensive. The price difference accounts for the savings between the H-piles and the pipe piles. It was assumed for these optimizations that the toe resistance model would be the same, regardless of pile type. In this case, because the piles are founded in sandy material, this may not be the case in the field.

Halifax County Bridge, Project B-2980

This bridge spans Beech Swamp on US 301/NC 481. It is a long bridge, consisting of nine spans with square prestressed concrete piles supporting the interior bents and H-piles supporting the exterior bents. A sketch of the interior pile bent is included in Appendix A.

General Information

Designed: 2004

Spans: 9 (35, 40 and 50 feet)

Skew: None

Interior Bents: 8

Pile Type: Eight 18-inch square prestressed, precast concrete piles; 40 to 50 ft long; Bent 1 is a 1:8 battered A-frame, but all piles in Bents 2 to 8 are vertical.

Free Pile Length: ~8 feet

Bent Cap: 39 inch wide by 30 inch deep Class A concrete beam

Cap Reinforcement: Five #9 bars (top) and four #9 bars (bottom)

End Bents: 2

Pile Type: Seven HP12x53; 50 ft long; Three brace piles battered 1:4

Free Pile Length: None

Bent Cap: 33 inch wide by 30 inch deep Class A concrete beam with wing walls

Superstructure: 15 standard 3 ft by 1.75 ft prestressed concrete cored slab units

Interior Bents Super/Substructure Connection: Two rows of 15 elastomeric bearing pads (Type I and II, 1 inch thick)

End Bent Super/Substructure Connection: One row of 15 elastomeric bearing pads (Type I, 1 inch thick)

Bearings:: Interior bents 1-7 have one fixed and one expansion bearing. End bents are fixed, and interior bent 8 has two expansion bearings.

Section Analysis

The section analysis for the 50 inch wide by 39 inch deep concrete bent cap is summarized in Figure 23 in terms of the moment curvature relationship obtained using hand calculations and the King Program (Priestley and Park, 1986). The ultimate capacity and cracked moment of inertia given the as-built dimensions and reinforcement are indicated on the figure. The cracked moment of inertia was used in the linear cap models for both SAP and MultiPier.

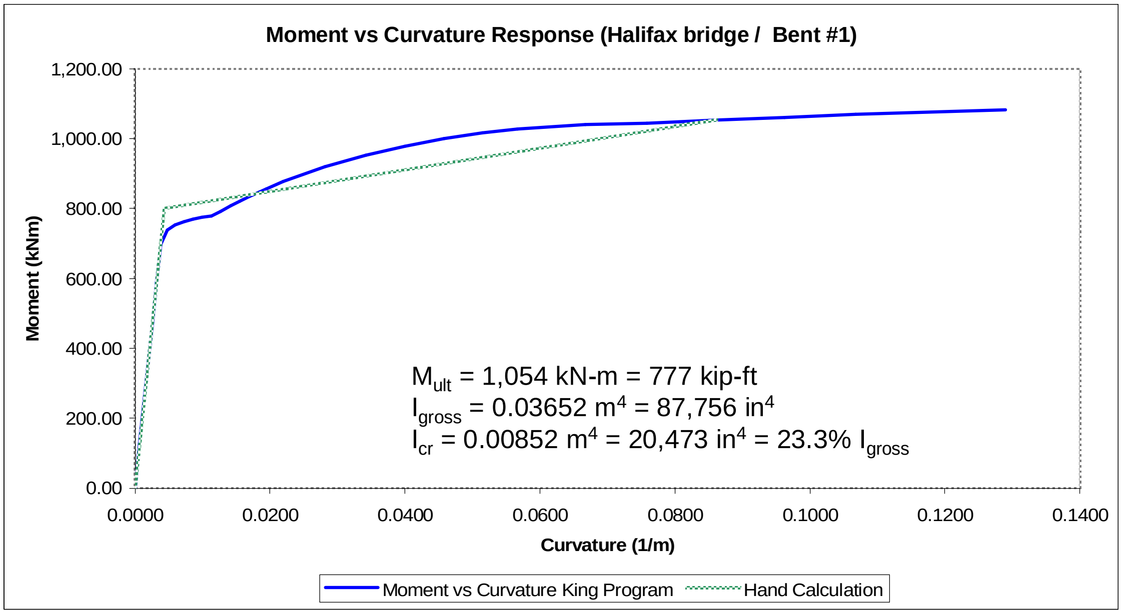


Figure 23. Halifax County Moment Curvature Analysis

Geotechnical Summary

For this analysis, one of eight interior bents was modeled. The design of the interior bent foundation was based on four borings spaced between the two 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 foot) coarse sand. Sandy clays and silts with N-values less than 6 blows per foot extend from depths of approximately 5 to 25 feet. N-values increase to between 20 and 30 blows per foot in the Cape Fear formation, where the piles apparently terminate.

Since the profile was relatively uniform across the site, Bent 2 was selected for the model. It should be mentioned that this bent was more representative of the majority of the interior bents, and did not include alternating batter piles like Bent 1.

Similar to previous bridges, the Halifax county bridge model assumed the pile toe was fixed. In this case, however, the piles were end bearing in silty clay, which may or may not be partially weathered rock. As mentioned earlier, in the absence of experience with both pile displacements under the pseudo-dynamic live loads and without confidence in the toe model provided by MultiPier in North Carolina's soils, a fixed toe was deemed appropriate for this bridge, if not ideal.

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

Analysis Results—SAP and MultiPier

The models for the Halifax County bridge were entered into MultiPier and SAP as shown in Figures 24 and 25, respectively. The input files are included in the Electronic Appendix. Table 11 shows the maximum displacements at a pile top and maximum moment generated in the bent cap for this bridge. Again, the pile toe was effectively fixed.

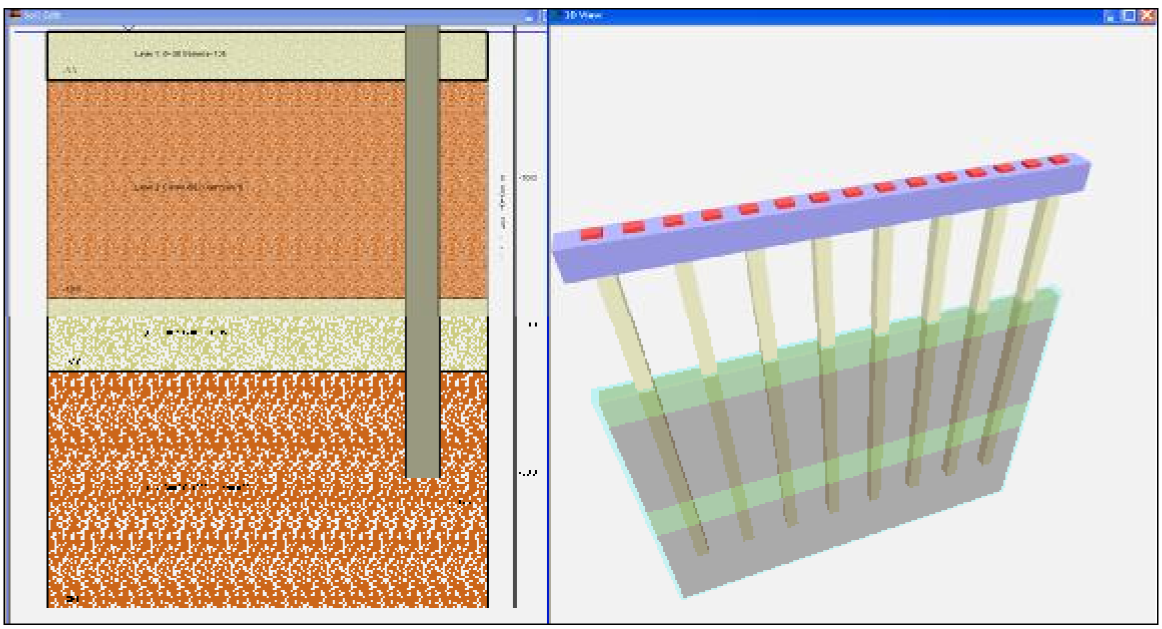


Figure 24. MultiPier Model--Halifax County Bridge

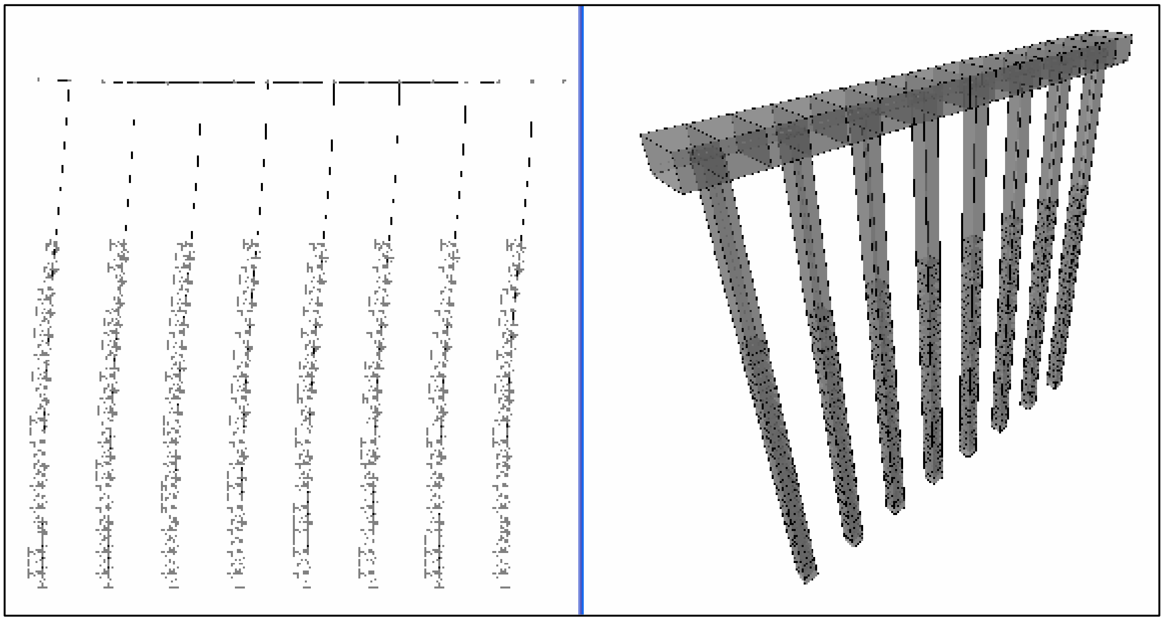


Figure 25. SAP Model—Halifax County Bridge, Deformed and Unloaded Shapes

Table 11. Halifax County Bridge, Nonlinear Analysis Results

Model	Axial Disp (in)	Transverse Disp (in)	Longitudinal Disp (in)	Maximum Moment in Bent Cap (kip-ft)
SAP	0.11	0.24	0.75	184
MultiPier	0.10	0.14	0.58	181
AASHTO Group	1A-LL3 Pile 6	2-LL1 Pile 1	2-LL5 Pile 4	1A-LL8

Analysis Results—Optimization

Table 12 shows the results of reducing the number of prestressed concrete piles or changing the concrete piles to H-Piles. For this bent, eight 18” square prestressed concrete piles were installed. The bent cap was 39 inches wide by 30 inches deep and reinforced with five #9 bars on the top and four #9 bars on the bottom of the cap.

Table 12. Halifax County Bridge, Reducing Number and Size of Piles

	18" PSC 8 piles	18" PSC 7 piles	18" PSC 6 piles	HP14x73 8 piles
Demand/ Capacity Ratio (Piles)	0.29	0.34	0.39	0.34
Transverse Displacement (pile top, in)	0.14	0.16	0.20	0.45
Longitudinal Displacement (pile top, in)	0.58	0.69	0.84	1.35
Axial Displacement (pile top, in)	0.10	0.12	0.14	0.24
Bent Cap Moment Max (kip-ft)	181	221	282	212
Minimum Rebar (Top, Bottom)	5 #5 5 #5	5 #6 5 #6	5 #7 5 #7	5 #5 5 #6
Minimum Rebar (NCDOT Specs)	5 #9	5 #9	5 #9	5 #9
Cost ($) Per Linear Foot (interp, 2001-2004)	60 estimated	60 estimated	60 estimated	45.5

Based on these results, all optimization scenarios seem reasonable for reducing the foundation costs. Minimum reinforcement approaches the NCDOT minimum reinforcement requirements as the number of piles is reduced.

The cost per linear foot for the 18 inch concrete piles in 2004 was roughly estimated from the 2001 to 2004 bid averages for all prestressed concrete piles. Figure 26 shows the data used to develop an estimate of $60 per linear foot cost for prestressed concrete piles. The computed foundation cost savings per bent are shown in Table 13.

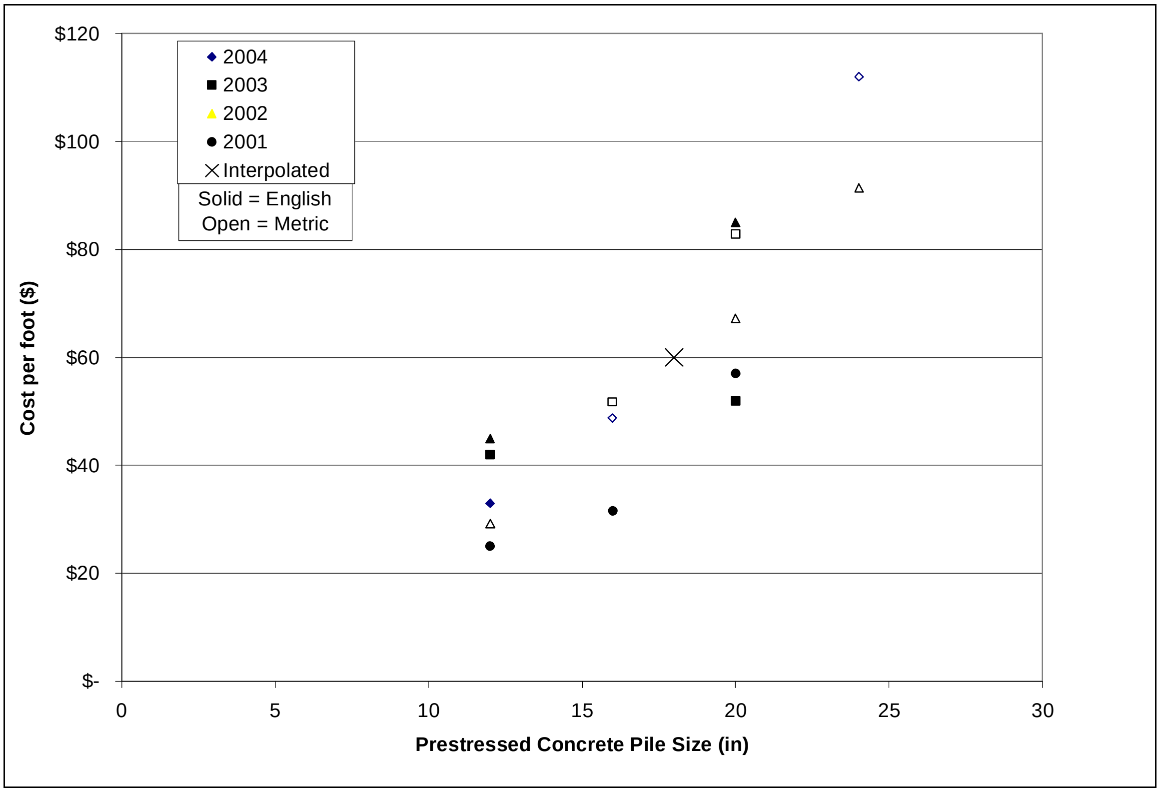


Figure 26. Estimated 18 inch Square PSC Pile Cost from Previous Years Bid Averages

Table 13. Halifax County Bridge, Estimated Foundation Costs per Bent

Layout	Total Pile Cost*	Cost Savings
Eight 18" PSC	$21,600	As Built
Seven 18" PSC	$18,900	$2,700
Six 18" PSC	$16,200	$5,400
Eight HP14x73	$16,380	$5,220

Washington County Bridge, Project R-2548B (Westbound Lane)

This bridge spans Old Millcreek on US 64. There are two bridges running parallel to one another—one for the eastbound lane and one for the westbound lane. The Westbound Lane bridge was the focus of these analyses, and, as the original design was submitted with metric units, metric units will be used here as primary units, with English units in parentheses. The bridge has three spans, with the two interior bents consisting of two rows of prestressed concrete piles and a concrete cap. A sketch of the interior pile bent is included in Appendix A.

General Information

Designed: 2000

Spans: 3 of 28, 19 and 19.25 m (91.8, 62.3 and 63.2 ft)

Skew: None

Interior Bents: 2

Pile Type: Two rows of five 406 mm (16 inch) square prestressed, precast concrete piles; 13 and 16 m (42.7 and 52.5 ft) long; Piles are spaced 600 mm (23.6 inches) center to center (1.5D). All piles are vertical.

Free Pile Length: ~2 m (~6.6 ft)

Bent Cap: 1520 mm (59.8 inches) wide by 910 mm (35.8 inches) (minimum) deep Class A concrete beam

Cap Reinforcement: Six #29 (metric) bars (similar to #9 bars in English Units) in the top and bottom of the cap

End Bents: 2

Pile Type: 13 prestressed concrete piles 305 mm (12 inch) square ; 13 or 15 m (42.6 or 49.2 ft) long; Six brace piles battered 1:4

Free Pile Length: None

Bent Cap: 940 mm (37 inch) wide by 760 mm (30 inch) (minimum) deep Class A concrete beam with wing walls

Superstructure: Five AASHTO Type IV Prestressed Concrete Girders, Continuous with live loads; prestressed concrete deck panels

Interior Bents Super/Substructure Connection: Two rows of five elastomeric bearing pads (Type IV, 52 mm thick); Diaphragms at each bent between girders

End Bent Super/Substructure Connection: One row of five elastomeric bearing pads (Type IV, 52 mm thick)

Joints: Interior bents are fixed joints. End bents are expansion joints.

Section Analysis

The section analysis for the 1520 mm (59.8 inch) wide by 910 mm (35.8 inch) deep concrete bent cap described above is summarized in Figure 27 and presented in terms of moment curvature analysis using hand calculations and the King Program (Priestley and Park, 1986). The ultimate capacity and cracked moment of inertia, given the as-built dimensions and reinforcement are indicated on the figure. The cracked moment of inertia was used in the linear cap model for SAP. Because the MultiPier model was created using a bent cap that is one-half the width of the actual cap in order to accommodate the configuration of two rows of piles, the cracked moment of inertia was halved, as well.

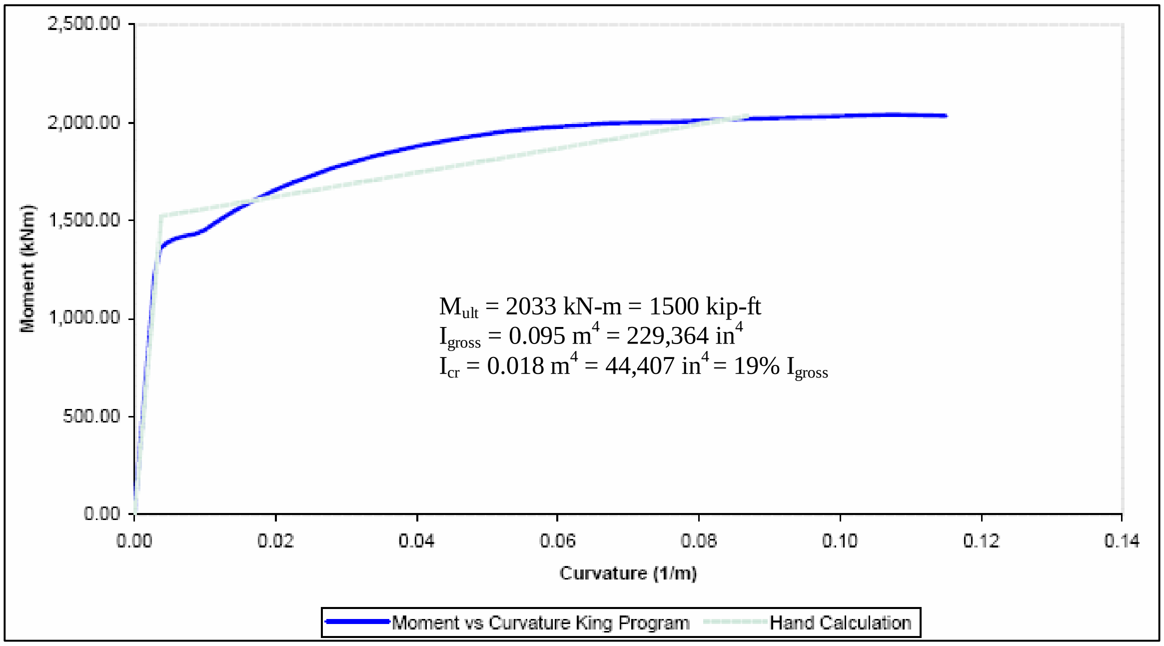


Figure 27. Washington County Moment Curvature Analysis

Geotechnical Summary

For this analysis, one of two interior bents was modeled for the westbound lane bridge. The interior bent foundation design was based on two borings at the bent location. The two borings were reasonably similar, and labeled Left Lane B1-A and Left Lane B2-A. Bent 1 and its corresponding soil profile (LLB1-A) was selected to create the MultiPier model. The water table was observed at, or very near, the ground surface.

In general, the two borings show approximately 1 m (3.3 ft) of muck and soft organic soils, underlain by silty clayey sands with organics. The Top of the Yorktown Formation was identified approximately 8 m (26.2 ft) below the ground surface. This sandy material has a range of N-values from 5 to 35 blows per foot. The piles were founded in the deeper, more competent material at depths of approximately 13 m (42.7 ft) below the ground surface, where N-values ranged from 31 to greater than 100 blows per foot.

Boring LLB1-A indicated approximately 1 m (3.3 ft) of dredge spoil, which had an N-Value of 3 blows per foot. The next 3 m (9.8 ft) consisted of silty clayey sand with organics, with N-values ranging from 13 to 16 blows per foot. From 4 to 9 m (12.1 to 29.5 ft), coarse sand with gravel was encountered, with N-values of 12 to 18 blows per foot.

Below 9 m (29.5 ft), material identified as the Yorktown Foundation was encountered. This consisted of 3 m (9.8 ft) of silty clayey calcareous sand with N-values of 21 and 35 blows per foot. A similar 1.5 m (4.9 ft) thick, lower N-value layer was observed before the boring was terminated in silty calcareous sand whose N-value increased with depth from 31 to 72 blows per foot. The boring was terminated 15.8 m (51.8 ft) below the ground surface; the piles were expected to terminate in this deeper, more competent layer.

As for the case of the Robeson bridge, the pile toe resistance model was assumed to be fixed. The soils encountered at this site are likely Partially Weathered Rock. Because of the uncertainty in how best to model this material, a nearly fixed toe model was assumed.

Lateral group analysis considered the spacing between the piles, which for this bridge was 2.56 meters (8.4 ft), or more than six times the 406 mm width of the pile (6D). Since the MultiPier manual does not give guidance beyond 5D, it was assumed for these analyses that the group behavior of these piles would be unaffected based on the larger spacing of 6D. All P-y multipliers for axial and lateral group loading, were 1.0.

Analysis Results—SAP and MultiPier

The bridge was modeled in MultiPier and SAP. Figures 28 and 29 show the input screens while the input files are included in the Electronic Appendix. Using symmetry, only half of the Washington County Bridge Bent was modeled in MultiPier using a single row of piles and a bent cap of one half the actual width of the as-built bent cap. This approximation was adopted because MultiPier's Pile Bent model is only set up for a single row of piles. MultiPier's Pier Cap model allows multiple rows of piles, but then, in such a case, the allowable concrete bent cap geometry is quite limited. For example, in the Pier Cap model, the moment of inertia is not directly specified in linear models and nonlinear models are not available.

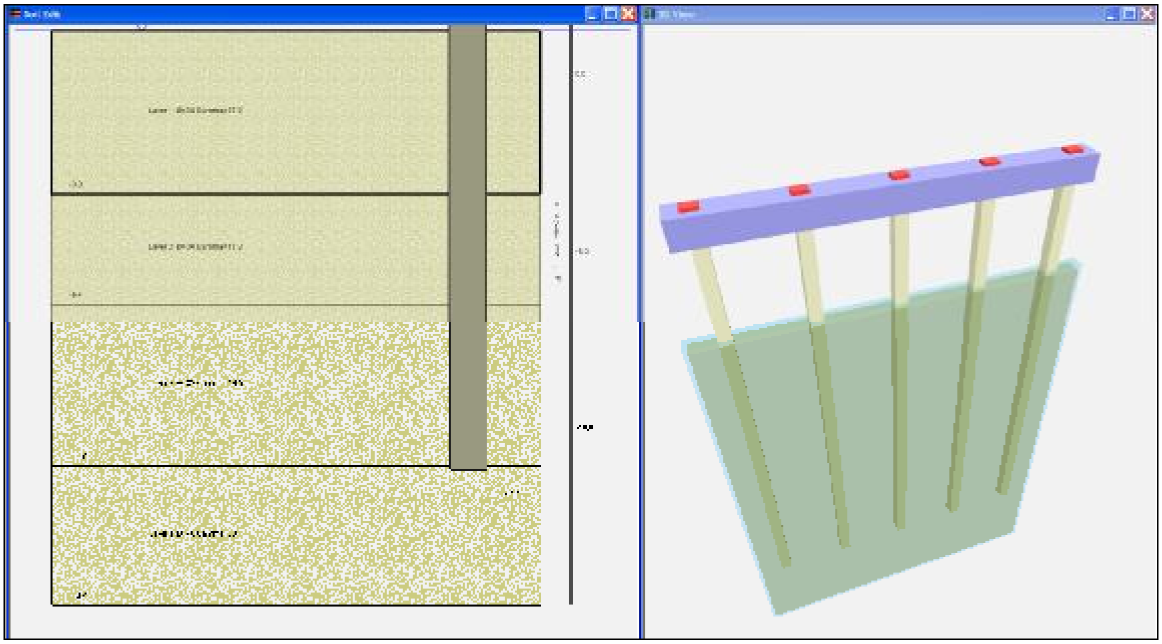


Figure 28. MultiPier Model--Washington County Bridge, One Row of Piles

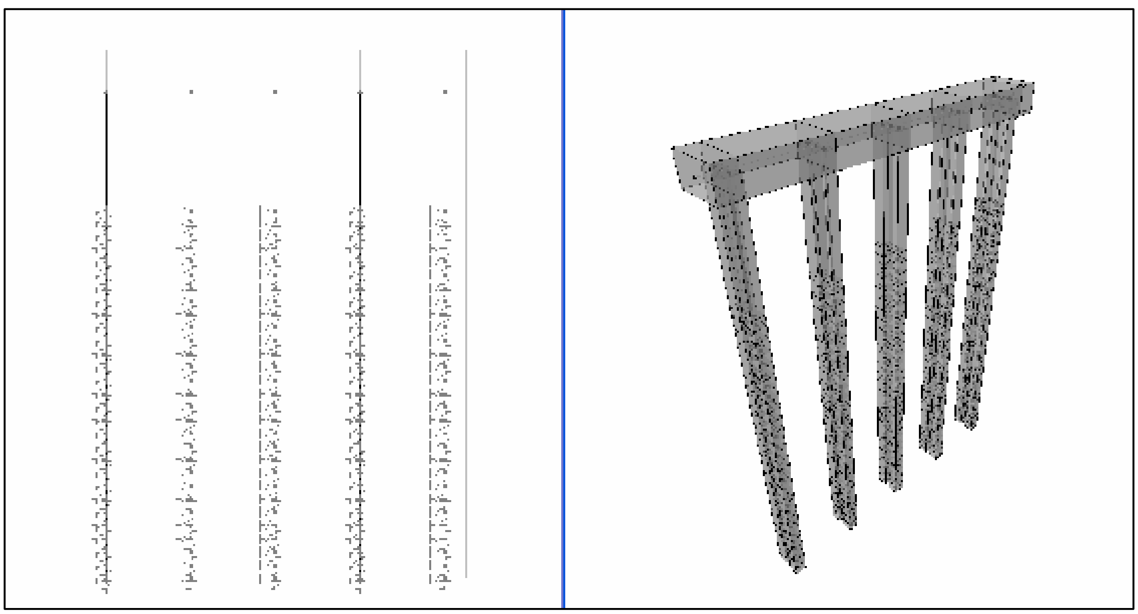


Figure 29. SAP Model--Washington County Bridge, Two Rows of Piles

To check the effects of this modeling approximation, the SAP model was created according to the as-built design. In order to create the MultiPier half-model, a spring with infinite rotational stiffness was placed at each bearing location to restrain rotation about the transverse axis, which approximates the effect of the other half of the bridge. Once the models were run, it was noted that the longitudinal displacements predicted by MultiPier were much lower than those predicted by SAP. When the cap was allowed to rotate longitudinally about the transverse axis, the predicted longitudinal displacement was closer to the estimated value using SAP.

Table 4.11 shows the maximum displacements at a pile top and maximum moment generated in the bent cap for this bridge. Because the loads were halved in MultiPier, the generated forces predicted in the piles by MultiPier should be correct. The moments predicted in the cap beam, however, should be doubled to take into account the other half of the bent cap and the other row of piles. This doubling is shown in Table 4.11 in the Pier cap moment section.

The actual longitudinal rotational stiffness offered by the other half of the bridge is however more likely to be somewhere between the two cases (fixed and free). This stiffness is partially dictated by the spacing between the two pile rows. In this bridge, the piles are spaced apart 600 mm (2 ft), or 1.5 times the pile's width. As the spacing gets closer to a single row of piles, the rotational stiffness likely approaches zero. Given a spacing of approximately 1.5D, the bent is probably rotationally acting more as a single row of piles than a widely spaced double row of piles or in a free head condition.

Table 14. Washington County Bridge, Nonlinear Analysis Results

Model	Axial Disp (mm; **in**)	Transverse Disp (mm; **in**)	Longitudinal Disp (mm; **in**)	Maximum Moment in Bent Cap (kN-m; **kip-ft**)
SAP	7.6; **0.30**	6.6; **0.26**	11; **0.43**	354; **261**
MultiPier (free long. rotation)	7.1; **0.28**	6.4; **0.25**	14; **0.55**	163*2 = 326; **240**
AASHTO Group	1A-LL3 Pile 6	2-LL1 Pile 1	2-LL5 Pile 4	1A-LL8

Analysis Results—Optimization

MultiPier was used to check the effects of reducing the number or size of the piles. For this bridge, ten 406 mm (16 inch) prestressed concrete piles were driven in two rows of five piles each. The bent cap was 1520 mm (59.8 inch) wide by 910 mm (35.8 inch) deep and reinforced with six #29 (metric, approximately #9 English) bars along both the top and bottom of the cap. The optimization results are summarized in Table 4.12.

First, a single row of five or six prestressed concrete piles was attempted, as this seemed as an obvious cost saving choice. In this case, MultiPier would not converge, indicating either excessive displacements or structural failure in one or more elements.

Table 15. Washington County Bridge, Reducing Number and Size of Piles

	406 mm PSC 10 piles	406 mm PSC 8 piles	HP14x73 Two row 10 piles	Single Row, 5 or 6 PSC
Demand/ Capacity Ratio (Piles, unitless)	0.31	0.56	0.35	Did Not Converge
Transverse Displacement (pile top, mm; **in**)	6.4; **0.25**	8.4; **0.33**	14; **0.55**	
Longitudinal Displacement (pile top, mm; **in**)	14.3; **0.56**	20.1; **0.79**	27.7; **1.09**	
Axial Displacement (pile top, mm; **in**)	7.1; **0.28**	12.3; **0.48**	12; **0.47**	
Bent Cap Moment Max (kN-m; **kip-ft**):	151; **111**	537; **396**	188; **139**	
Metric Rebar (Top, Bottom)	6 #14	6 #26	6 #16	
	6 #16	6 #29	6 #20	
Minimum Metric Rebar (NCDOT)	6 #29	6 #29	6 #29	
Cost Per Linear Meter (2004); **per linear foot**	$160; **$51**	$160; **$51**	$149.30; **$45**	$160; **$51**

The pile cost per linear meter included in Table 4.12 was obtained directly from the 2004 NCDOT bid averages. An estimated total foundation cost and estimated cost saving is included in Table 4.13. In 2004, the prestressed concrete piles were surprisingly only marginally more expensive than the HP14x73.

Table 16. Washington County Bridge, Estimated Foundation Costs per bent

Layout	Total Pile Cost*	Cost Savings
10 406mm (16 in)PSC	$25,600	As Built
8 406mm (16 in) PSC	$20,480	$5,120
10 HP14x73	$23,886	$1,714

Summary

The results presented in this chapter have shown that the MultiPier model can be approximately reproduced in 3-D SAP program. This essentially verifies the output results from MultiPier model and inspires confidence in its use. The primary difference observed between the two programs was in the longitudinal and transverse displacements. As discussed in Chapter 3, such a difference is likely due to approximation in the reproduction of the nonlinear lateral soil spring load-displacement (P-y) curves input for SAP to match those used in MultiPier. Due to the relative coarseness of the MultiPier-produced data points, the SAP P-y curves are slightly less stiff at lower loads than MultiPier. This lower stiffness means higher displacement obtained from SAP as indicated by analysis results.

Similarly, there is difference in results between the MultiPier and SAP models for Washington bridge. MultiPier does not model double rows of piles in a way similar to single rows. A single row model, assuming symmetry in the longitudinal direction, was used in MultiPier. The differences observed between SAP, in which the bridge was modeled as-built, and the simplified MultiPier model were primarily in the longitudinal direction. It seems the pile top rotation in the longitudinal direction should be represented by a range of unrestrained to restrained conditions as the spacing between rows increases.

An optimization analysis was conducted for each bridge by reducing the number, or size, of the piles. In all cases, it was shown that some savings in material and installation costs can be realized using the nonlinear analysis. The Northampton bridge optimization also showed that moments in the cap beam can be reduced significantly if bearing locations are located directly above the piles.

The use of a full nonlinear pile bent analysis program could be somewhat complicated. In the next chapter, possible equivalent models using a more simple point of fixity approach will be considered, as a similar model is currently in use by the NCDOT. This method allows the designer to model the bridge as a frame, and consider only certain parameters based on performing “single” pile geotechnical analysis.

CHAPTER 5: EQUIVALENT LINEAR MODEL FOR ANALYSIS AND DESIGN

Background

For design purposes, it is a common practice to use an elastic equivalent model to approximate a more complex nonlinear soil-pile system (Figure 30a). In the elastic model, the piles are considered fully fixed at some depth below ground surface and the soil is ignored (Figure 30b). The embedded length, also called depth to fixity, L_f, is estimated from formulas or from the results of a nonlinear lateral single-pile analysis. The resulting equivalent model is especially convenient for the design engineer, who can model a single pile as a cantilever and a pile bent as a frame. In both cases, the piles are fixed at the specified point of fixity. The results of this simplification are straightforward structural computations that can provide all required information for design.

Over the past 40 years, several procedures have been proposed for the estimation of L_f values, such as those proposed by Davisson and Robinson (1965), and Y. Chen (1997). The current practice of NCDOT was summarized in Chapter 2 and three criteria were found to be used in the assessment of the depth to fixity: Davisson and Robinson's equations as will be described later in this chapter, the point of maximum negative moment, and primarily the point of maximum negative deflection. The later two are from the results of a nonlinear lateral single-pile analysis performed using the computer program LPILE, or any other code using the P-y method. It is shown later in this chapter than none of these criteria yield an equivalent model capable of fully predicting single pile response.

In this chapter, a new approach to defining an equivalent model from the results of a nonlinear lateral single pile analysis is proposed. Assessment of suitability and applicability of the proposed method is presented based on the analyses results of the four case studies described in Chapter 4.

Davisson and Robinson's model for computing L_f

In general, L_f is evaluated either from simplified formulas or from the results of nonlinear lateral soil-pile analysis. The most often used L_f equations are those proposed by Davisson and Robinson in 1965. These equations have been incorporated into the AASHTO LRFD Bridge Design Specifications (2004) and their use is recommended for the assessment of buckling effective length only. For piles in clays, L_f is evaluated from

Equation 5.1, and for piles in sand from Equation 5.2. In both cases, L_f is measured from the ground level, as shown in Figure 30 (b).

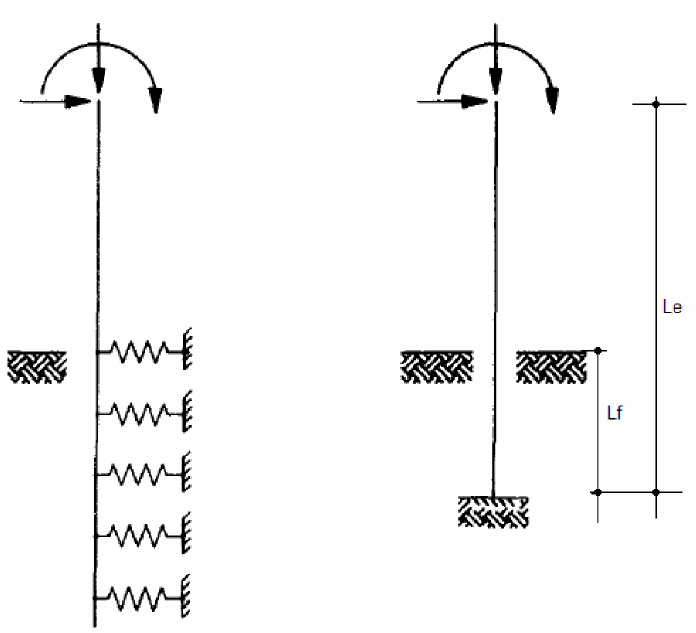


Figure 30. Soil-pile models. (a) Nonlinear soil-pile model (b) Equivalent Model

$$L_f = 1.4\left[\frac{E_p I_{py}}{E_c}\right]^{0.25} \qquad \text{(Clay)} \qquad (5.1)$$

$$L_f = 1.8\left[\frac{E_p I_{py}}{n_h}\right]^{0.20} \qquad \text{(Sand)} \qquad (5.2)$$

In these two equations, E_p and I_{py} are the elastic modulus and inertia of the pile, E_c is the elastic modulus for clays (see Table 17 for representative values) and n_h is the rate by which the soil modulus increases with depth in sands (see Table 18 for representative values). Equations 1 and 2 are based on beam on elastic-foundation theory and assume a long, partially embedded pile in a single uniform layer of either clay or sand. The coefficients value of 1.4 in Equation 5.1 and 1.8 in Equation 5.2 are set so the model can approximately match bending and buckling response simultaneously but results from these models will not match lateral stiffness for deformation purposes.

A drawback of this approach is that, for a multiple soil layer profile, the engineer has to determine an equivalent soil layer of either sand or clay in order to use the equations.

Other problems are that these models do not distinguish between free-headed and fix-headed piles and that they cannot be used to assess lateral displacements.

Table 17. Representative E_c values for clays after Y. Chen (1997)

Clay type	s_u (tsf)	E_c (tsf)
Soft	0.25	16.75
Medium	0.47	31.4
Stiff	0.81	54.4
Very stiff	1.47	98.5

Table 18. Representative n_h values for sands after Y. Chen (1997)

Sand type	Saturation condition	n_h (tsf/ft)
Loose	Moist/dry	30
	Submerged	15
Medium	Moist/dry	80
	Submerged	40
Dense	Moist/dry	200
	Submerged	100

As mentioned previously, the NCDOT currently uses an elastic model for the design of pile bents. In this model, the effect of the soil is taken into account by fixing the piles at a point below ground surface (point of fixity, POF). The main disadvantage of the elastic model used by the NCDOT arises from the fact that the point of fixity is not located such that the results from the model matches the response of the nonlinear soil-pile system.

Equivalent elastic model from single pile nonlinear lateral analysis

The following proposed procedure yields an equivalent system that predicts the response of a nonlinear soil-pile model under a specific level of loading. he method determines the length and section modification factors such that a pile fixed at a designated point of fixity responds the same as a pile embedded in soil when similar loads are applied. This is achieved by studying the results of a nonlinear lateral analysis performed for the pile-soil system.

The boundary conditions at the top of the pile must be taken into account when evaluating the equivalent model parameters. Two limiting conditions may occur: the head of the pile is allowed to translate and rotate (free-head condition) or the head of the pile translates without rotation (fix-head condition).

The application of the proposed procedure requires the calculation of the following parameters as shown in Figure 31:

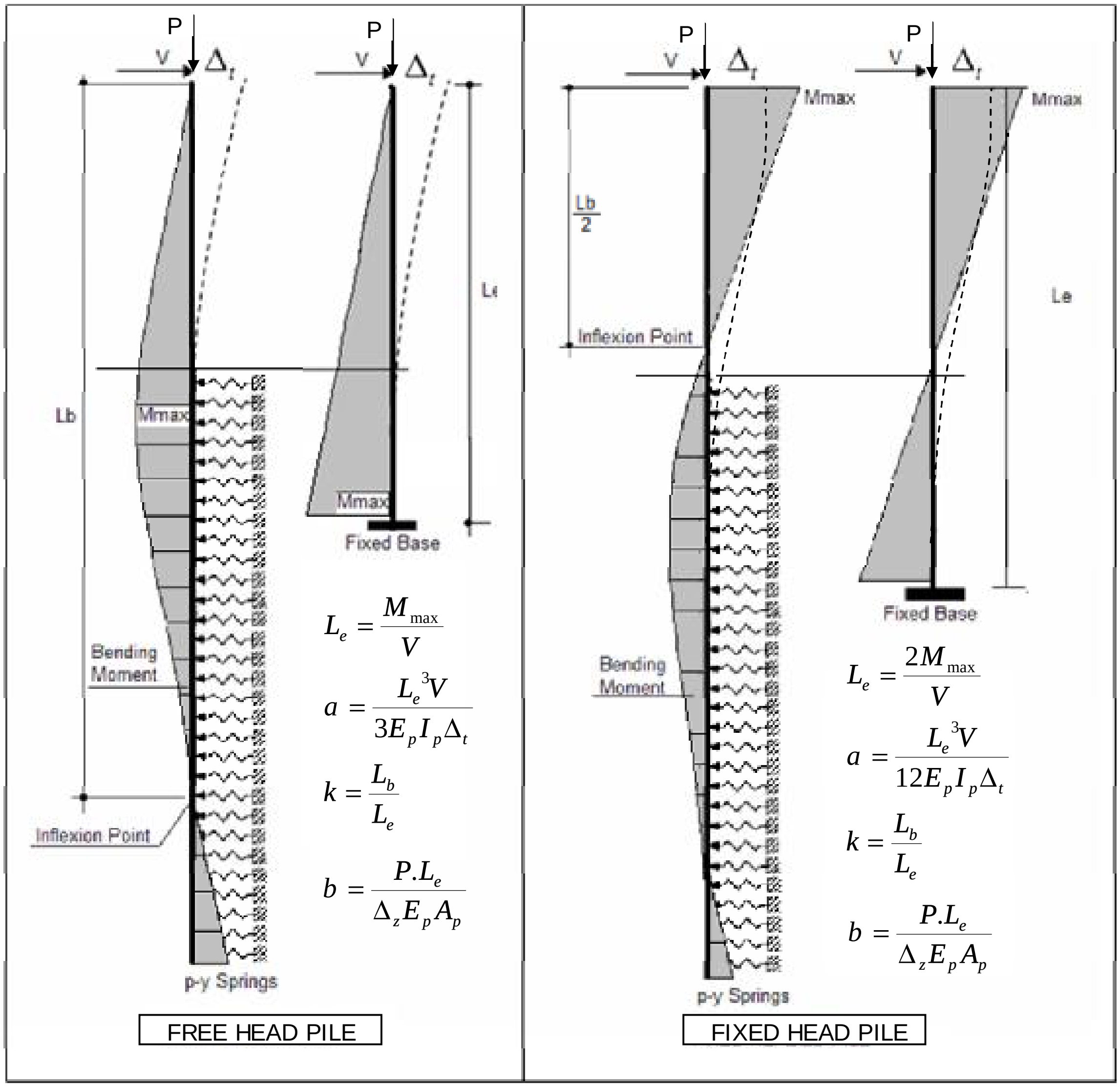


Figure 31. Equivalent model parameters

For a free head condition , the equivalent model parameters can be evaluated as follows:

$$L_e = \frac{M_{max}}{V} \tag{5.3}$$

$$a = \frac{L_e^3 V}{3E_p I_p \Delta_t} \tag{5.4}$$

$$k = \frac{L_b}{L_e} \tag{5.5}$$

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

$$L_e = \frac{2M_{\max}}{V} \tag{5.6}$$

$$a = \frac{L_e^3 V}{12 E_p I_p \Delta_t} \tag{5.7}$$

$$k = \frac{L_b}{L_e} \tag{5.8}$$

For both, free head and fixed head condition:

$$b = \frac{P.L_e}{\Delta_z E_p A_p} \tag{5.9}$$

Where:

L_e The 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.

a 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. 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 Displacement at the top of the pile caused by the application of the lateral load 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. It is 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 that is applied to the area of the pile in the equivalent model to result in the same axial deformation as the nonlinear soil-pile model under the effect of the axial load, P.

A_p Area of the pile section.

P Applied axial load

Δ_z Axial displacement of pile top

Recommended procedure for computing the equivalent model parameters:

1. Perform a nonlinear lateral single pile analysis as described in Chapter 3. In the analysis, representative properties of the soil and pile shall be used. If some properties/conditions differ from pile to pile 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 of the soil and pile so the response of the single pile is roughly representative of the bent. Alternatively, the worst case scenario can be used.
2. Apply boundary conditions at the top of the pile (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 pile, and determine the top displacement (Δ_t), the maximum moment (M_{max}) and the location of inflection points as shown in Figure 31.
4. Compute the equivalent model parameters using Equations 5.3–5.9 and Figure 31.
5. Build, analyze, and design an elastic model using the geometry, section properties and calculated equivalent model parameters. This might require the use of a frame analysis software such as RCPier or others.

The proposed equivalent model has several advantages over a model based on L_f equations:

- The equivalent model will yield deformation level similar to that obtained from a nonlinear soil-pile system under a specific level of loading.
- If second order effects (P-Δ) were accounted for in the nonlinear analysis, moment and deformation magnification will be automatically reflected in the response of the equivalent model.
- Multi layered soil profiles can be taken into account.

Equivalent model parameters for NCDOT bridge case studies

Using the proposed procedure, the equivalent model parameters were computed for Robeson, Northampton, Halifax, and Washington bridges. A summary of the results is shown below.

Robeson County Bridge.

The interior bent of the Robeson County Bridge has eight HP 14x73 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 ft. 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. Following the recommended procedure, a single pile model was created using MultiPier (Figure 32). Results from the nonlinear single pile analysis, as well as the input data for the calculation of the equivalent model parameters, are shown in Figures 33 and 34 for a fixed head and free head condition respectively. The equivalent model parameters were calculated and are show in Table 19. Figures 33 and 34 also show the location of the point of maximum negative moment and point of maximum negative deflection. These points, as indicated previously, have been used by NCDOT as points of fixity.

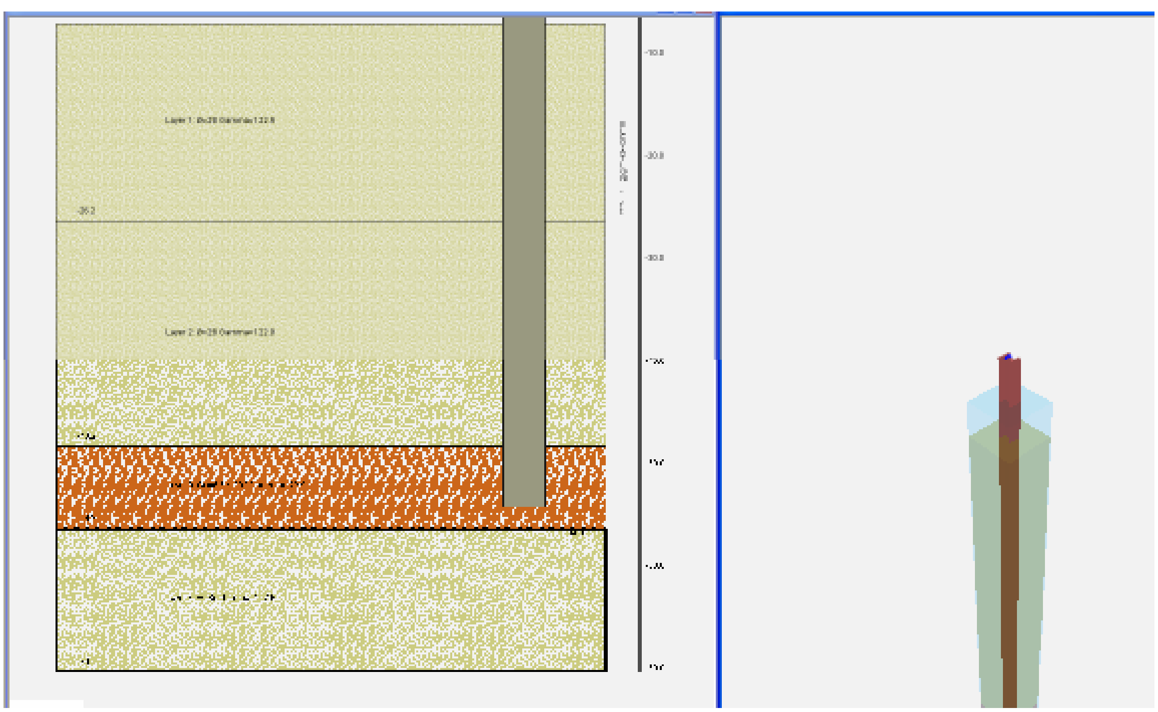


Figure 32. Nonlinear single pile model for Robeson Bridge

Table 19 Equivalent model parameters for Robeson Bridge

EQUIVALENT MODEL PARAMETERS				
HEAD	Le (ft)	**a**	**b**	k
FIXED	14.5	0.95	0.26	1.1
FREE	10.8	0.29	0.20	2.2

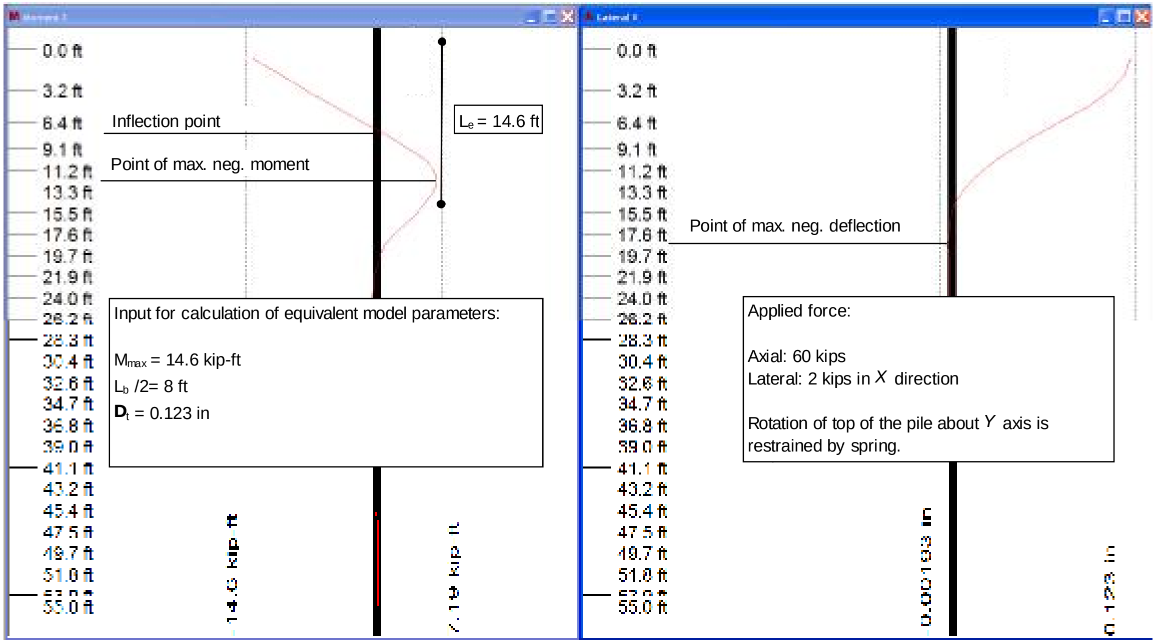


Figure 33. Results of nonlinear single pile analysis for Robeson Bridge. Moment and deflection diagram for fixed head condition

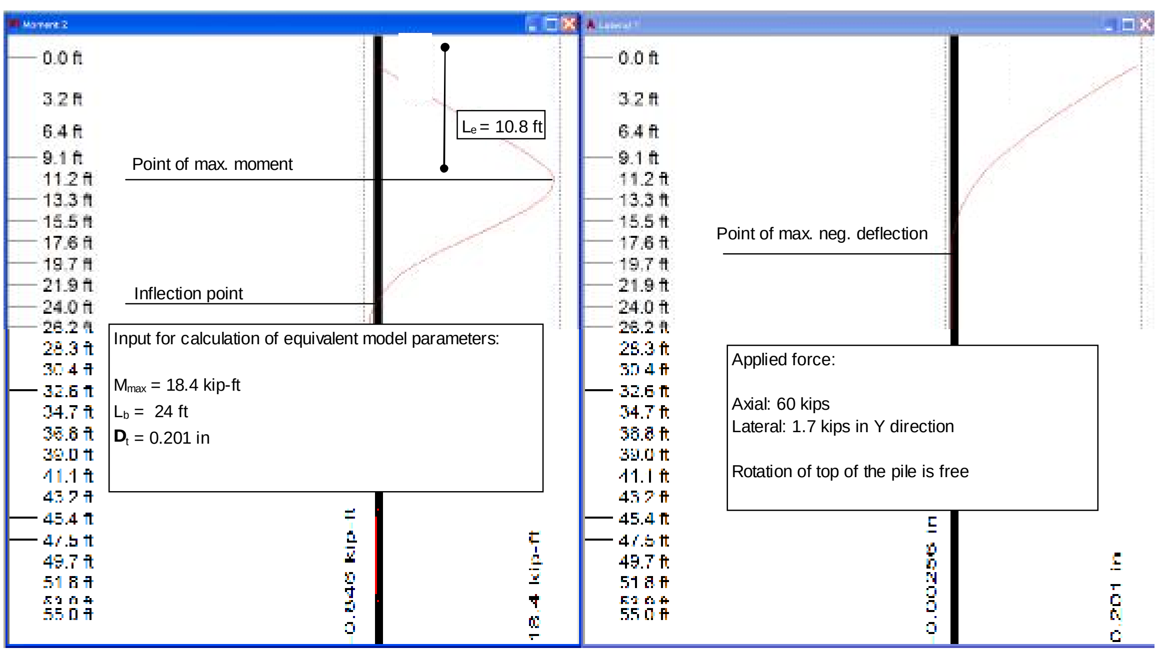


Figure 34. Results of nonlinear single pile analysis for Robeson Bridge. Moment and deflection diagram for free head condition

Northampton County Bridge

The interior bent of the Northampton Bridge has five 24-inch 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 ft. 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. Results from the nonlinear single pile analysis as well as the input data for the calculation of the equivalent model parameters, are shown in Figures 35 and 36 for fixed and free head conditions, respectively. The equivalent model parameters were calculated and are show in Table 20.

Table 20. Equivalent model parameters for Northampton Bridge

EQUIVALENT MODEL PARAMETERS				
HEAD	Le (ft)	**a**	**b**	k
FIXED	22.2	0.94	0.37	1.2
FREE	16.2	0.37	0.27	2.1

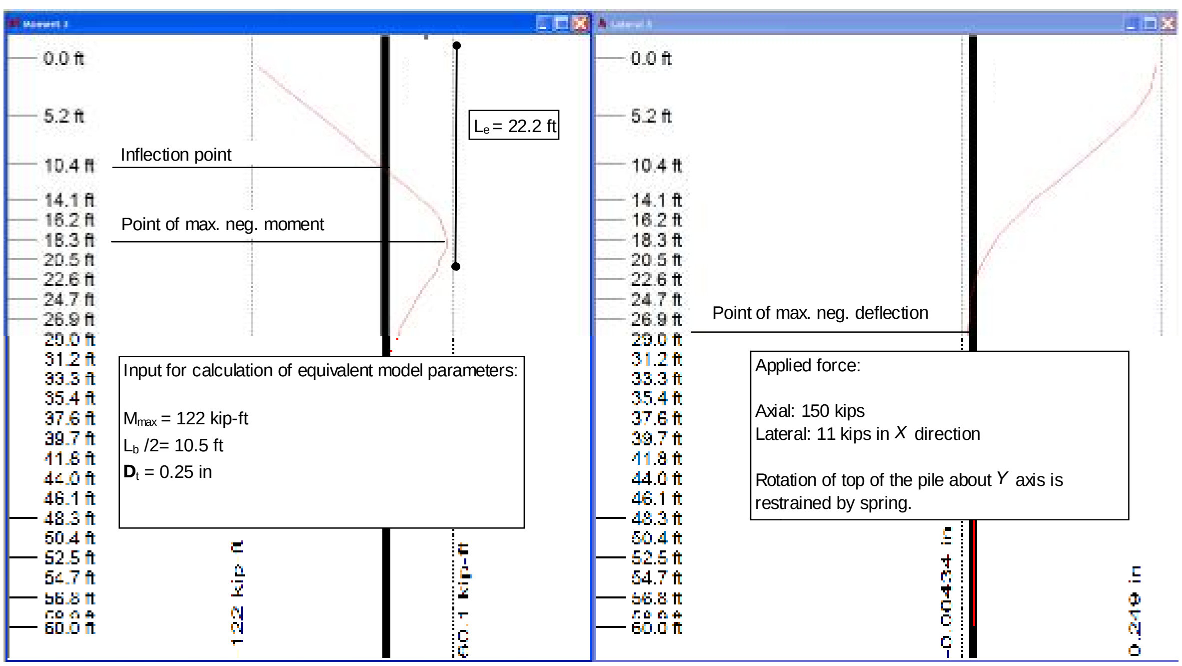


Figure 35. Results of nonlinear single pile analysis for Northampton Bridge. Moment and deflection diagram for fixed head condition

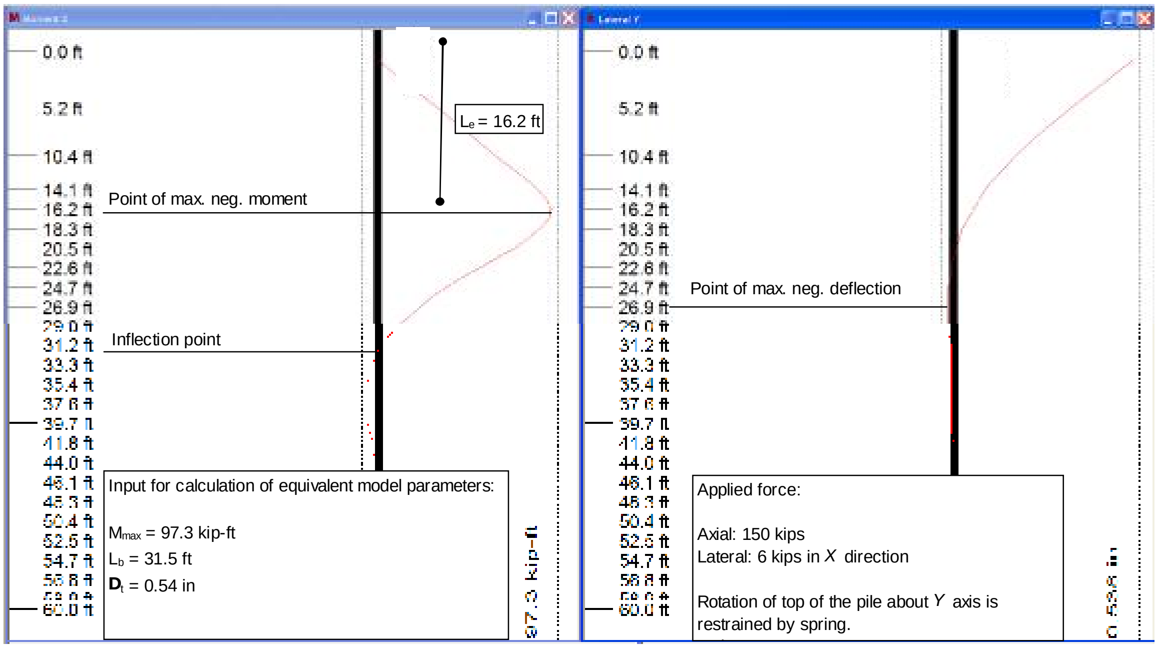


Figure 36. Results of nonlinear single pile analysis for Northampton Bridge. Moment and deflection diagram for free head condition

Halifax County Bridge.

The interior bent of the Halifax Bridge has eight 18-inch square PSC piles. Each pile has a flexural stiffness *EI* of 38622420 kip-in^2. The total length of the piles is 45 ft. 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. Results from the nonlinear single pile analysis, as well as the input data for the calculation of the equivalent model parameters, are shown in Figures 37 and 38 for fixed and free head conditions, respectively. The equivalent model parameters were calculated and are shown in Table 21.

Table 21. Equivalent model parameters for Halifax Bridge

EQUIVALENT MODEL PARAMETERS				
HEAD	Le (ft)	**a**	**b**	k
FIXED	22.6	0.98	0.57	1.0
FREE	16.8	0.42	0.41	1.8

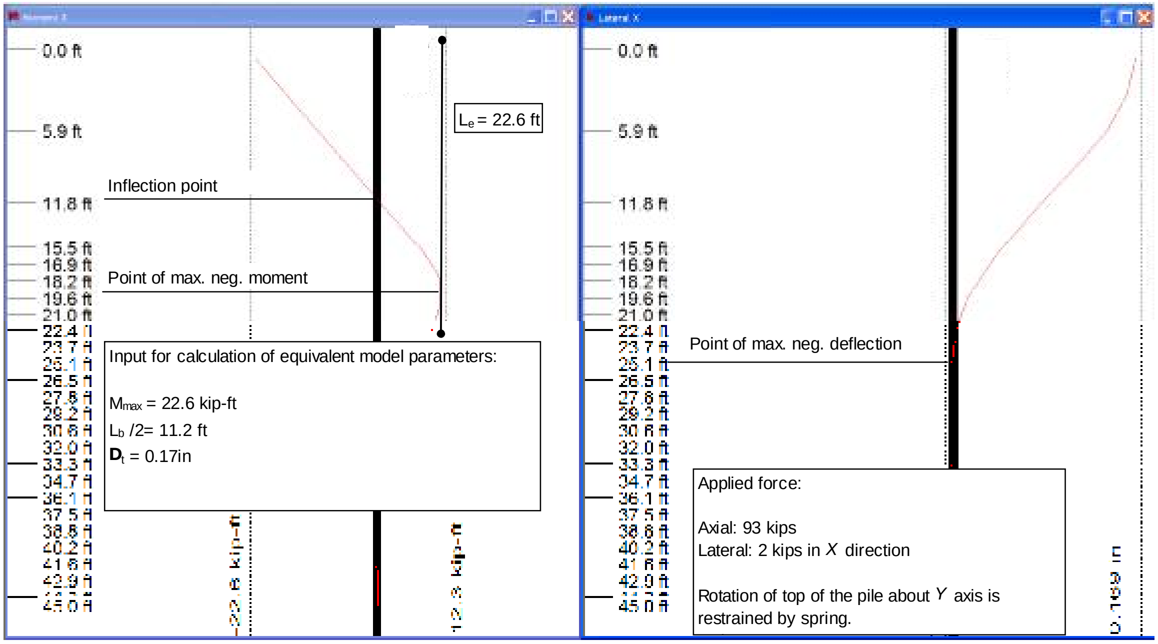


Figure 37. Results of nonlinear single pile analysis for Halifax Bridge. Moment and deflection diagram for fixed head condition

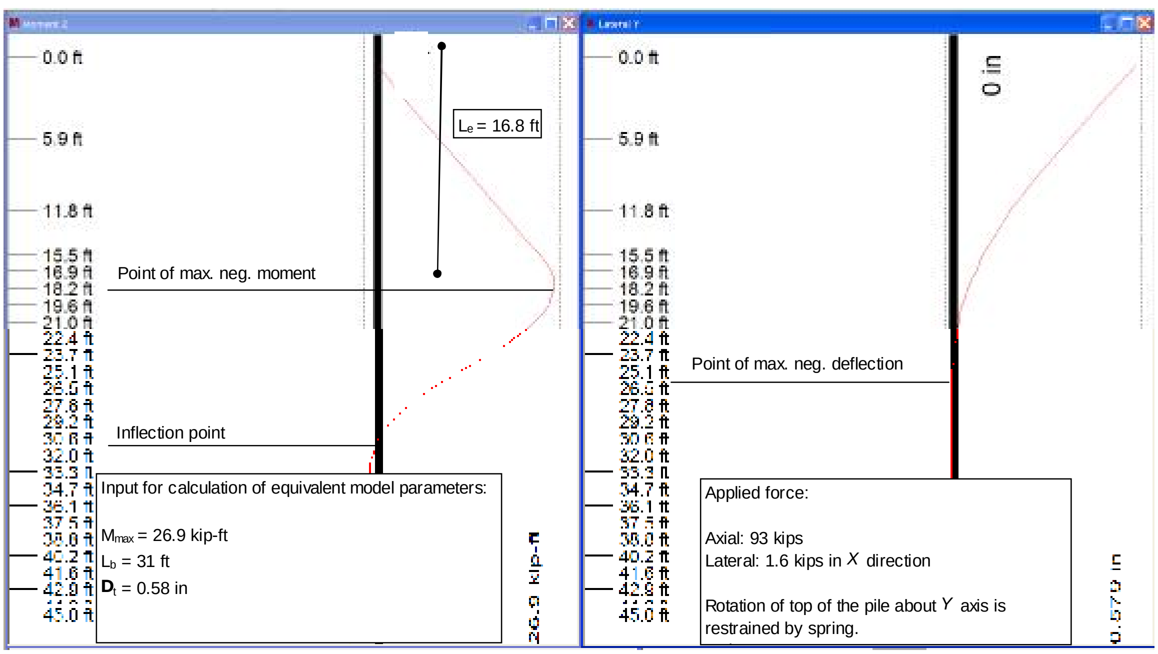


Figure 38. Results of nonlinear single pile analysis for Halifax Bridge. Moment and deflection diagram for free head condition

Washington County Bridge

The interior bents in the Washington Bridge have two rows of five 406 mm square PSC piles. Each pile has a flexural stiffness *EI* of 68,924 kN-m^2. The total length of the piles is 15.3 m. The anticipated factored axial dead load per pile is 500 kN. The anticipated factored lateral load acting in the transverse direction is 24 kN per pile and the anticipated factored lateral load acting in the longitudinal direction is 12 kN per pile. Results from the nonlinear single pile analysis, as well as the input data for the calculation of the equivalent model parameters, are shown in Figures 39 and 40 for fixed head free head conditions, respectively. The equivalent model parameters were calculated and are showed in Table 22.

Table 22. Equivalent model parameters for Washington Bridge

EQUIVALENT MODEL PARAMETERS				
HEAD	Le (m; ft)	α	β	k
FIXED	6.28; **20.60**	1.06	0.41	1.0
FREE	4.81; **15.78**	0.47	0.31	1.7

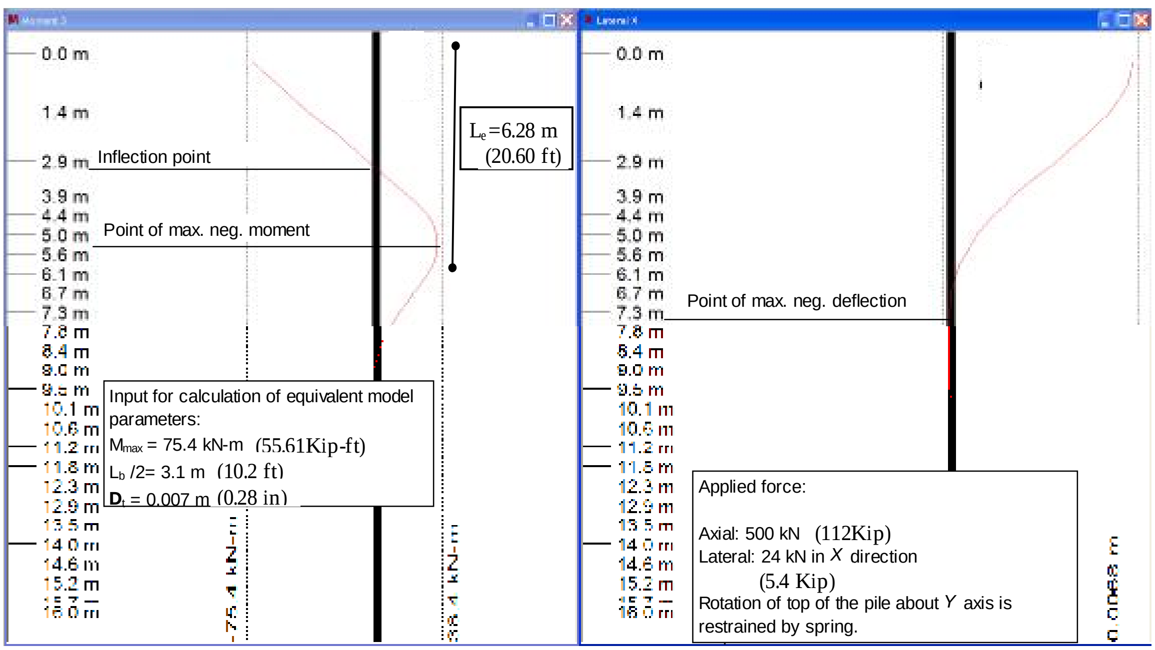


Figure 39. Results of nonlinear single pile analysis for Washington Bridge. Moment and deflection diagram for fixed head condition

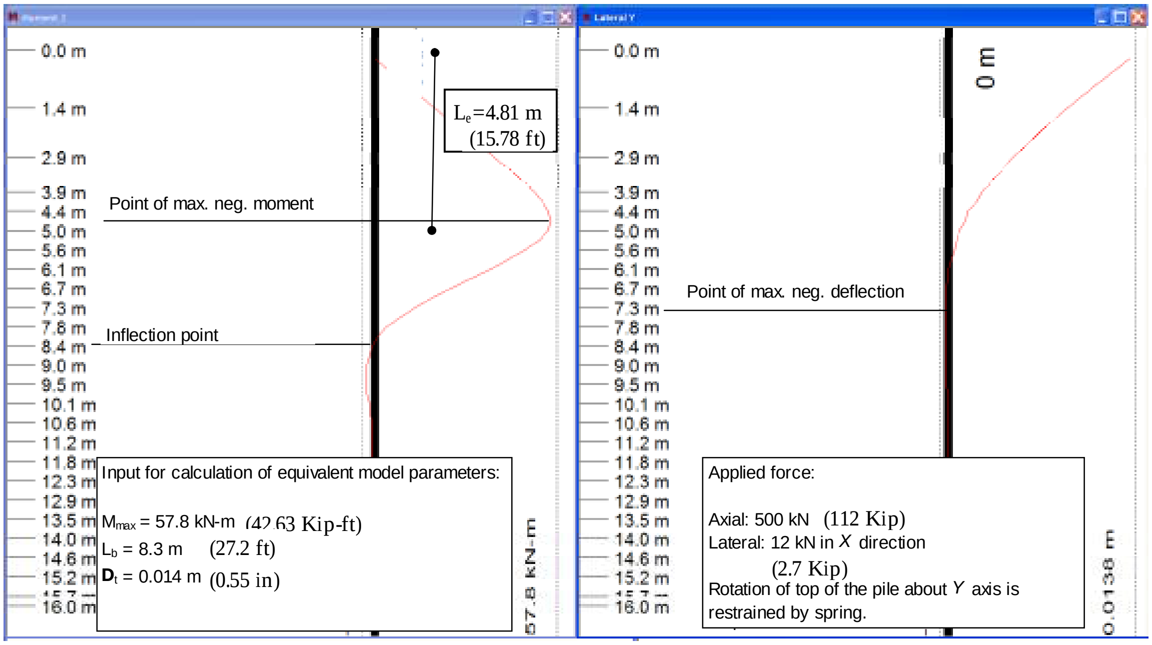


Figure 40. Results of nonlinear single pile analysis for Washington Bridge. Moment and deflection diagram for free head condition

Sensitivity of the equivalent model parameters to the applied load

As explained previously, the equivalent model parameters are 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 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, a and k to the level of axial and lateral loading applied to a nonlinear single-pile model.

Figures 41 to 44 show how the equivalent model parameters change due to the level of applied load and the corresponding top displacement. For each bridge, three levels of axial load representing 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 either caused the analysis to not converge after 100 iterations or excessive lateral displacement. Once each analysis was performed, the equivalent model parameters where computed using Equations 5.3 to 5.8.

For all the studied bridges, the results of the sensitivity analysis show that:

- The stiffness of the soil-pile system degrades with the increase of lateral force.
- L_e tends to increase with the increase in lateral load magnitude.
- α tends to decrease with the increase in lateral load magnitude.

- k does not show a defined trend of change with the increase in lateral load magnitude.
- k ranged from 1 to 1.2 for fixed head condition and from 1.6 to 2.5 for the free head condition.
- α ranged from 0.4 to 1.08 for fixed head condition and from 0.23 to 0.60 for free condition.
- 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. It was also observed that the second order deflection caused by the axial load is counteracted by an increase in the inertia of the PSC piles in the Halifax and Washington bridges (Figures 43 and 44).

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

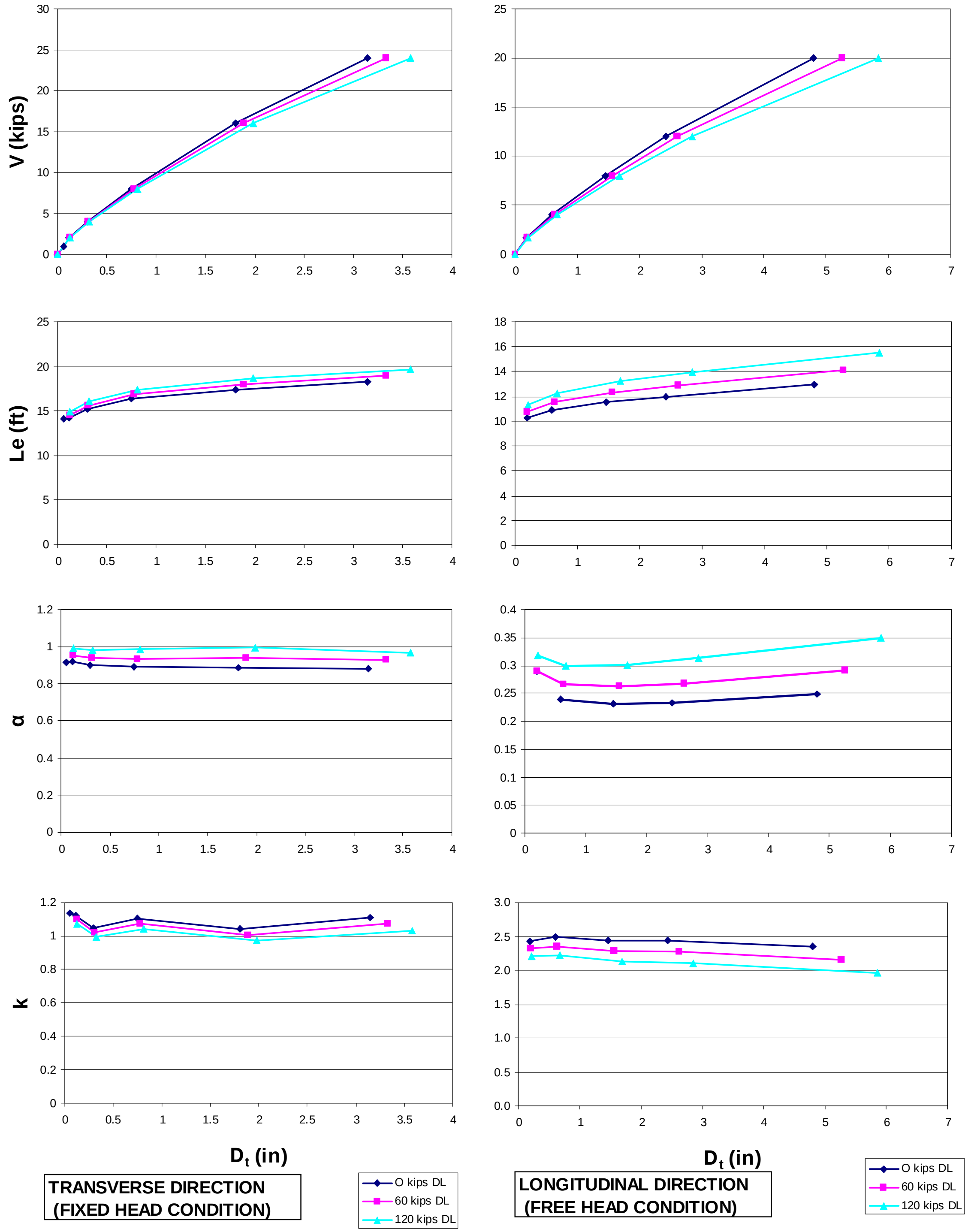


Figure 41. Sensitivity of equivalent model parameters for Robeson County Bridge

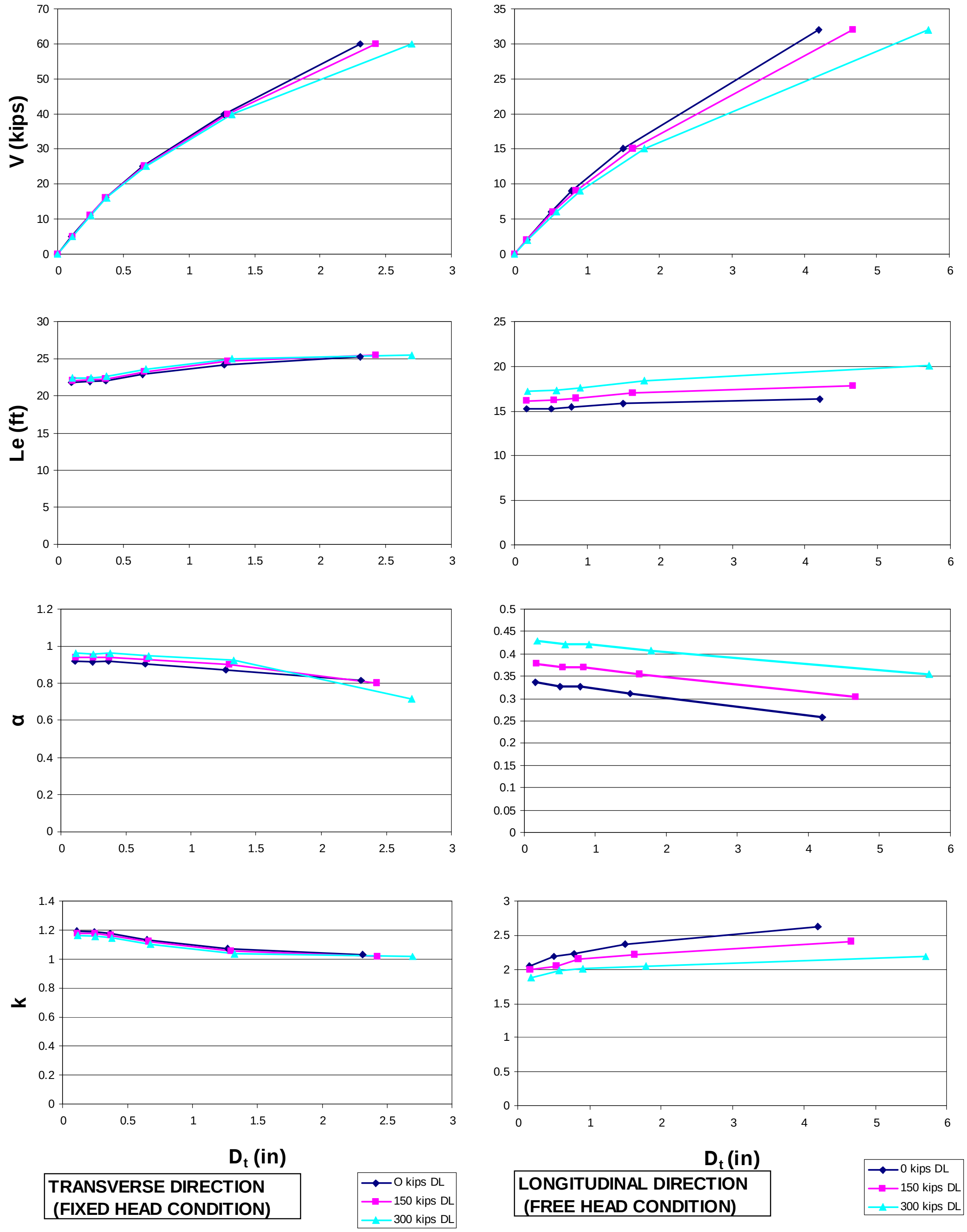


Figure 42. Sensitivity of equivalent model parameters for Northampton County Bridge

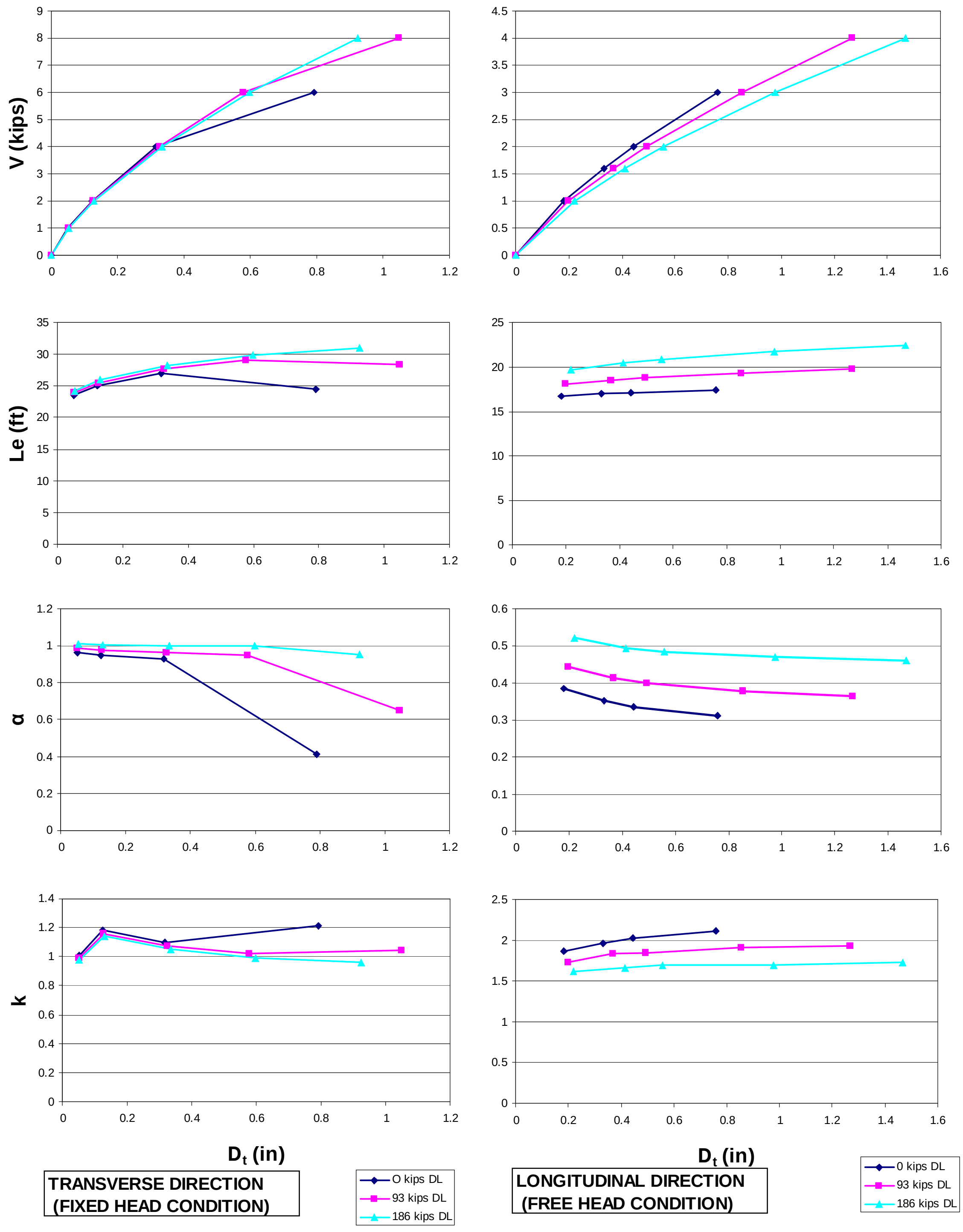


Figure 43. Sensitivity of equivalent model parameters for Halifax County Bridge

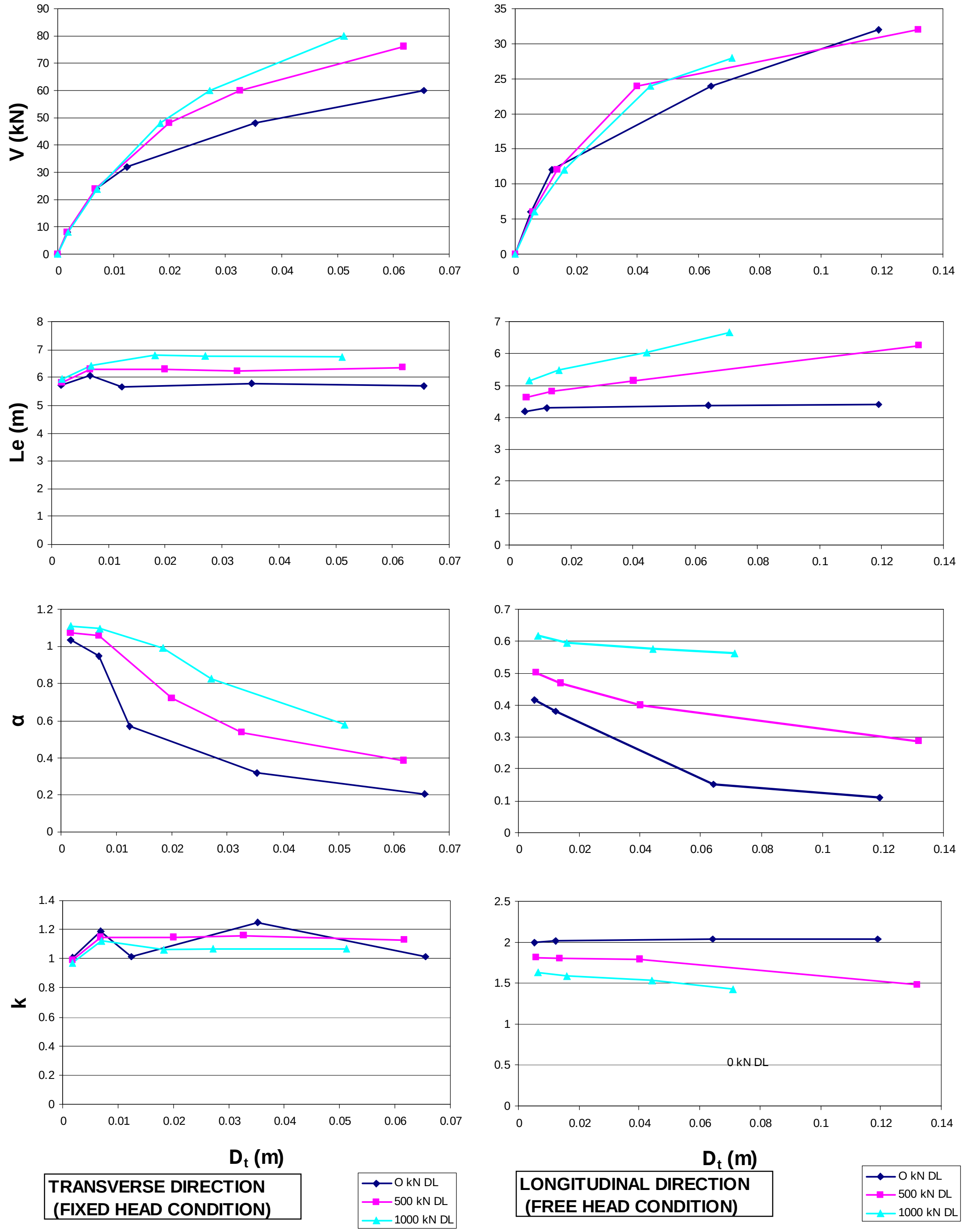


Figure 44. Sensitivity of equivalent model parameters for Washington Bridge

Comparison between proposed model and current practice

For the design of the interior bents in the four study bridges, NCDOT used an elastic model in which the lengths of the piles was assumed equal to the free length plus the depth to the point of maximum negative deflection. The point of maximum negative deflection was found by performing a nonlinear single pile lateral analysis. Table 23 summarizes the pile length used in Georgia Pier for each bridge, as well as the *k* values used to determine the axial-buckling capacity of the piles. These values will be referred to as DOT-POF models

Table 23. Summary of pile length and *k* values as used in Georgia Pier.

CASE STUDY	TRANSVERSE		LONGITUDINAL	
	HT*	k	HT*	k
ROBESON (ft)	21.69	1.2	21.69	2.10
NORTHAMPTON (ft)	31.55	1.2	31.55	2.10
WASHINGTON (ft)	25.92	1.2	25.92	2.10
HALIFAX (ft)	29.10	1.2	29.1	2.10

*HT is the length of the pile as it was used in Georgia Pier.

Table 24 shows a summary of the equivalent model parameters as obtained using the proposed equivalent models. Table 25 shows ratios between the displacement, maximum moment and buckling capacity predicted by the proposed equivalent model formulation and the models that were actually used for the design of the bridges. The comparison was made in terms of the response of a single pile, with the appropriate cross section, to which the length, section modifier parameters and boundary conditions were assigned.

Table 24. Summary of equivalent model parameters

CASE STUDY	TRANSVERSE			LONGITUDINAL		
	L_e	α	k	L_e	α	k
ROBESON (ft)	14.56	0.95	1.10	10.80	0.29	2.20
NORTHAMPTON (ft)	22.20	0.94	1.20	16.20	0.37	2.10
WASHINGTON (ft)	20.60	1.06	1.00	15.78	0.47	1.80
HALIFAX (ft)	25.50	0.98	1.00	18.53	0.41	1.70

Table 25. Performance of the proposed equivalent model formulation vs. the DOT-POF models

CASE STUDY	TOP DISPLACEMENT PREDICTED BY EQUIVALENT MODEL/DOT-POF MODEL		MAXIMUM MOMENT PREDICTED BY EQUIVALENT MODEL/DOT-POF MODEL		BUCKLING CAPACITY PREDICTED BY EQUIVALENT MODEL/DOT-POF MODEL	
	TRANSVERSE	LONGITUDINAL	TRANSVERSE	LONGITUDINAL	TRANSVERSE	LONGITUDINAL
ROBESON	0.3	0.4	0.7	0.5	2.6	3.7
NORTHAMPTON	0.4	0.4	0.7	0.5	2.0	3.8
WASHINGTON	0.5	0.5	0.8	0.6	2.3	3.7
HALIFAX	0.7	0.6	0.9	0.6	1.9	3.8

The proposed equivalent point of fixity model was developed to closely match the results of the full nonlinear single pile lateral analysis for a given loading case. To compare elastic cantilever models, the DOT POF and proposed equivalent (i.e. two point of fixity with reduction factor) models were applied to a single cantilevered pile (in an attempt to more simply approximate an LPILE analysis) in each of the four case studies. The results presented in Table 25 showed that in the transverse direction, the proposed equivalent formulation predicts on average 50% of the displacement, 70% of the maximum moment, and 220% more axial-buckling capacity than the DOT POF model. In the longitudinal direction, the proposed equivalent formulation on average predicts 50% of the displacement, 55% of the maximum moment, and 375% more axial-buckling capacity than the DOT POF model. These averages are for to the four study cases, and are specific soil conditions and pile types used in the analysis.

Pile design check from results of equivalent model analysis

An interesting feature of the proposed equivalent model formulation is that the resulting moments and displacements include P-Δ effects. This is the case since the equivalent model parameters were chosen to match the response of the nonlinear soil pile model in which material and geometric nonlinearities were taken into account.

According to AASHTO Bridge specifications, the design of reinforced concrete piles is based on a factored axial load and a magnified factored moment. The elastic analysis of the equivalent model will give magnified moments (i.e include P-Δ effects) so the design check can be done directly by using a short column interaction diagram. No k value will be needed other than for checking whether or not the column is slender.

In the case of steel piles, as in the reinforced concrete pile case, there is no need for moment magnification, but the use of the effective length factor *k* is required to evaluate the axial capacity of the pile as indicated in AASHTO Bridge design specifications.

Comparison of results between nonlinear and equivalent pile bent analysis

The results of the nonlinear analyses, discussed in Chapter 4, are compared with the results of elastic analyses. The elastic analyses were performed for the case study bridges using the proposed equivalent model formulation, as well as the depth to fixity and modeling technique originally used by the NCDOT design. The results of the analyses using the proposed equivalent formulation are noted in Tables 22 to 25 as “Equivalent Model”, and the results based on the point of fixity definition by NCDOT are noted as

"DOT-POF". The equivalent models were based on the equivalent parameters shown in Tables 18 to 21.

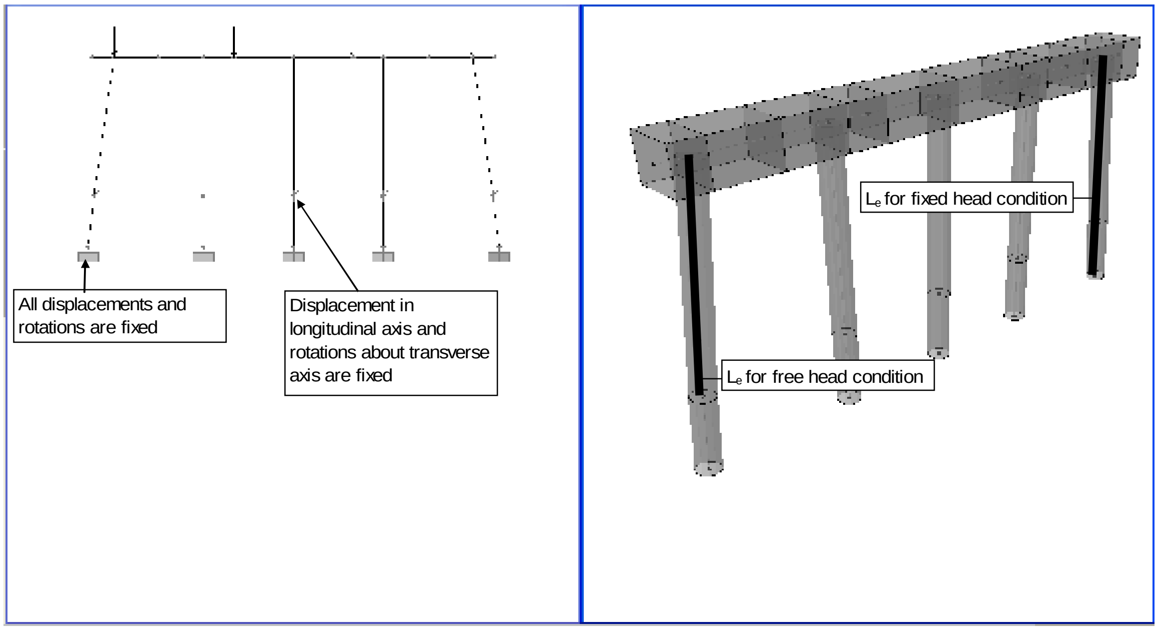


Figure 45. Elastic frame model with equivalent model parameters for Northampton County Bridge

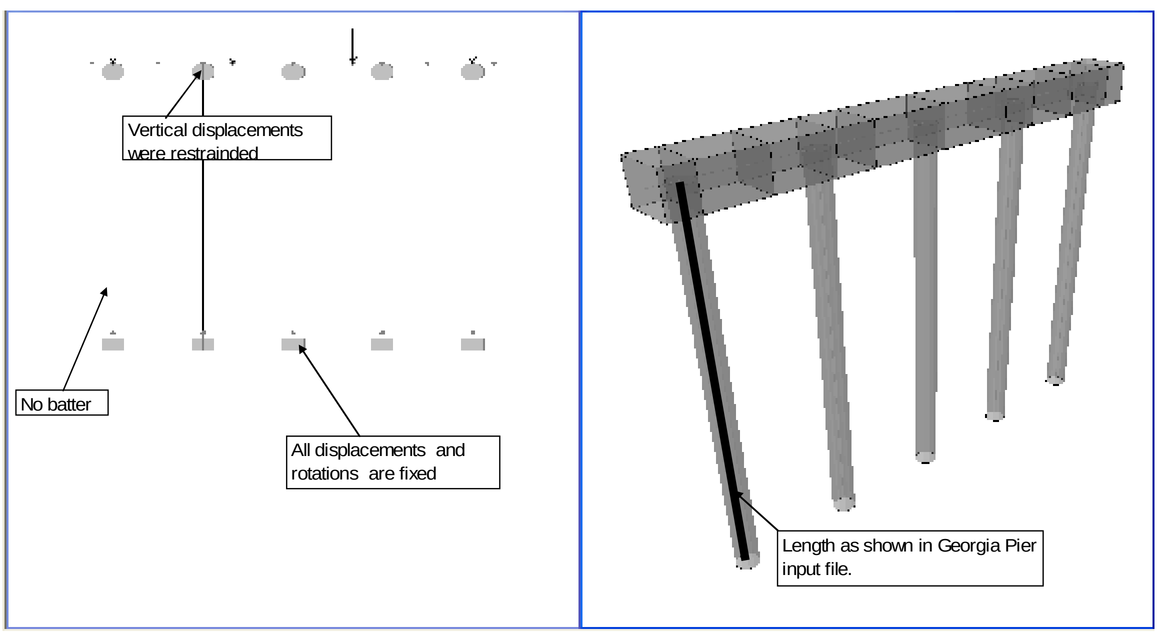


Figure 46. Elastic frame model with point of fixity as defined by NCDOT for Northampton County Bridge

For each bridge, the elastic analysis of a pile bent using the proposed equivalent model involved the generation of a 3D frame model using SAP 2000. The models had 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 accomplished simultaneously in the transverse and longitudinal directions by the creation of intermediate nodes and by assigning restraints to assure fixity of the appropriate degrees of freedom as can be seen in Figure 45.

The elastic analysis with the depth to fixity values as used by NCDOT (shown in the Georgia Pier input file for each pile bents under study) was accomplished by generating a 3D frame model in SAP. The same section properties, geometry and loading were used as in the corresponding nonlinear analysis. For example, Figure 46 shows how the model was defined in SAP for Northampton County Bridge. The length of the piles were shortened and fixed at the base and, the vertical displacement of the piles at the pile top was restrained as used in Georgia Pier.

Tables 26 to 29 show the results of nonlinear analyses, as well as the results of elastic analysis. For all bridges, the proposed elastic equivalent model properly predicts the response of the nonlinear models as solved using MultiPier. This is not surprising since the equivalent model parameters were chosen to match the results of a nonlinear lateral single pile analysis performed in MultiPier, in which soil, loading, and boundary conditions were model accordingly.

The elastic analyses based on the point of fixity definition and modeling techniques used by NCDOT (DOT-POF models) over predicted lateral displacements in pile bents, similar to the over prediction for a single pile analysis shown in Table 25. However, the moment predicted by the DOT-POF models relatively matched the values from the nonlinear analysis. This contradicts the results presented in Table 25, in which it was shown that, for a single pile analysis, the DOT-POF model would predict greater moments. The difference is mainly due to the restraint applied to the pile-cap joint nodes in the DOT-POF models which prevents these nodes from displacing vertically. In the nonlinear models, as well as in the proposed equivalent models, the piles were allowed to deform axially. In the case of unevenly distributed live load cases, small differential vertical movements in the pile-cap joints induce rotation and therefore additional moments in the cap beam and in the piles.

It is important to note that Georgia Pier does not compute displacements. The displacement values that NCDOT uses to check the designs come from the nonlinear lateral single-pile analysis performed to evaluate the point of fixity. Vertical displacements are treated separately if considered to be significant geotechnically.

Table 26. Equivalent model analysis results for Robeson Bridge (after Table 4)

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 27. Equivalent model analysis results for Northampton Bridge (after Table 8)

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 28. Equivalent model results for Halifax Bridge (after Table 11)

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	

Table 29. Equivalent model results for Washington Bridge (after Table 14)

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)	m22 (Kip-ft)
SAP (nonlinear)	0.30	0.26	0.43	261	176	35	7
MultiPier	0.28	0.25	0.55	240	182	50	1
Proposed Equivalent Model	0.28	0.26	0.40	177	185	35	4
DOT-POF model	0.00	0.53	1.07	18	187	32	0
Group	1A-LL4 Pile 4	2-LL1 Pile 1	2-LL5 Pile 5	1A-LL1	GIII LL1 - PILE 3		

Additional Considerations

This study has considered only the behavior of a single pile bent. It assumes all loads applied to a tributary area of the superstructure are distributed to the pile bent beneath it, and that there is little load in this tributary area transferred to, for example, the abutments or taken by the connections. It is likely that factors such as the stiffness of the connection between substructure and superstructure, the stiffness of the abutment, and the continuity of the bridge girders play a complicated role in the distribution of applied (particularly lateral) loads. An analysis approach that considers the entire abutment-superstructure-substructure-bearing pad system could result in different stresses and displacements compared to a free standing bent. .

Summary

Based on the model development and analysis results presented in this chapter, the following issues seem to contribute to the built-in conservatism in current NCDOT practice:

- The definition of a single deeper POF leads to prediction of larger displacements, moments and less axial capacity of the piles.
- Magnification of the moments to check the moment-axial interaction for piles even though the moments are already magnified through the use of the nonlinear single pile lateral analysis.
- Superstructure-substructure interaction is not accounted for. Superstructure and connections stiffness are very likely to reduce displacements on pile bents.

CHAPTER 6: LIMIT STATES

In order to ensure the stability and/or functionality of pile bents under loading demands, deformation due to applied forces, or other conditions should be limited. Abundant information, recommendations, and norms that limit undesirable levels of deformation under certain demands (such as service loads) are available for structural design of buildings. Unfortunately, the research reports and literature available for limiting deformation of bridge structures is scarce. Without appropriate criteria, knowledge regarding actual bridge performance is largely unkown during the design phase. For that reason, it is important to establish criteria to define limit states for lateral, vertical, and differential deformations.

Broadly defined, a limit state is defined by the occurance of an event. For example, cracking, crushing, yielding of reinforcement, and buckling of reinforcement in the case of Reinforced Concrete members, are all limit states. As many limit states occur well into the non-linear range of response, they are best defined on the basis of deformation quantities such as displacement, drift, curvature, or strain. The coupling of a limit state (say yielding of reinforcement) and a prescribed loading (say a 75 year return period earthquake) is commonly known as a performance level, and a series of performance levels constitutes a performance objective. (Bachman et al., 1999). These objectives will depend on the type of loading and on the severity of failure. Thus, for example, a 2 inch displacement may be acceptable under earthquake loading, but unacceptable under dead or live load conditions. Similarly, a separate set of limiting displacements may be applied for long term settlements in addition to limits on deformation under AASHTO loading cases.

This chapter considers a few performance levels that may result in significant displacements if they are not considered. A literature review is presented, followed by a preliminary series of analyses to consider geotechnical limitations, structural member level movements that may cause damage, and full scale bridge movement limits.

Current NCDOT Displacement Limits

Mainly three displacement types are considered for current NCDOT practice: total vertical displacements, differential vertical displacements within the bent, and lateral displacement. Total vertical displacements are only considered for geotechnical design; for structural design purposes they are assumed to be small. Differential vertical displacements between piles in a cap are not considered significant, in part because they are not expected due to the short duration of the live loading. The lateral displacements are based on single pile lateral analysis. Usually NCDOT uses an initial limit of one inch, although larger displacement has been accepted for some bridges.

Differential vertical displacements due to elastic pile compression from asymmetric loading was investigated in earlier chapters, and found to induce significant stresses in the concrete bent cap. However, vertical displacement of most pile foundations driven to partially weathered rock, rock or other dense soil strata is expected to be insignificant. Vertical displacements may become more of an issue for piles designed to be predominantly friction piles or piles driven in moderately dense sand layers.

Geotechnical Limit States

A study was undertaken to check how gap opening at the ground level due to the application of lateral loads will impact axial capacity of piles. For a flexible pile in soils with some cohesion, the gap could remain open for a period of time, which would effectively reduce the amount of skin resistance acting along the length of the gap. This would lead to a reduction in axial capacity (See Figure 47). In a cohesionless soil, the soil is expected to fill the created gap. This soil, however, would exert less horizontal pressure along the deformed length of the pile and the axial capacity of the pile may be reduced.

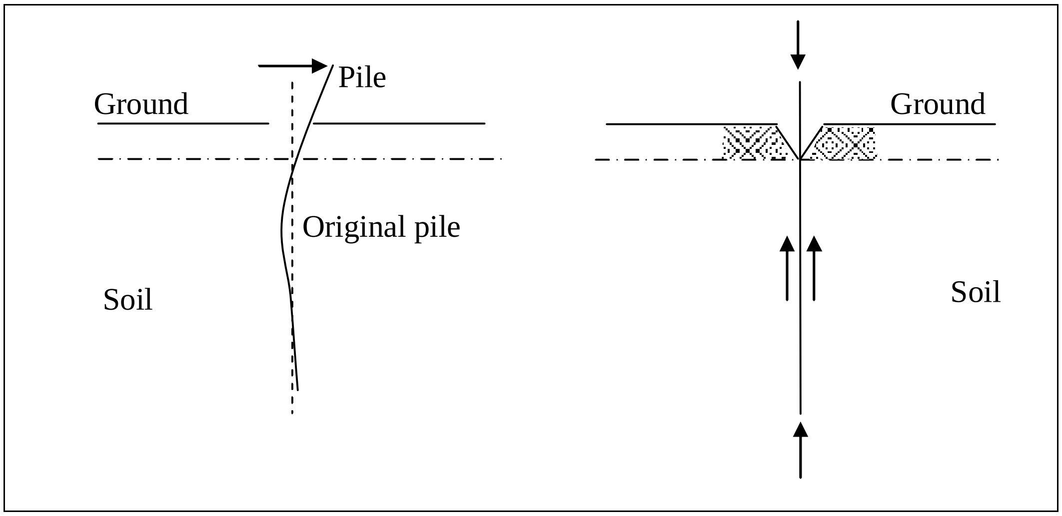


Figure 47. Gap formation and loss of shaft resistance--lateral load causing gap and vertical loading with gap

Meyerhof (1983) observed a similar reduction in axial capacity with tests on model piles. The mechanism for the reduction in those tests, however, did not seem to be the same as noted here. Meyerhof's model pile and theoretical studies considered both inclined loads and applied moments, as well as piles installed on a batter.

Piles in Sands with End Bearing

An analysis was performed to check the effect of lateral load on axial pile capacity. First, the MultiPier Robeson bridge model, which has predominantly sand along the length of the pile, was subjected to increasing lateral load applied at the top of each pile. The lateral load ranged from 1 to 5.5 kips in this analysis as greater lateral loads lead to no convergence. For each lateral load case, the deflected shape was plotted, as shown in Figure 48, and the deflection at the ground surface (i.e., the size of the gap opened between the trailing edge of the pile and the soil) and the point where the deflection's sign changed from positive to negative were recorded. The difference in depth between these two points was noted as the length of shaft resistance affected by the gap.

More likely, the reduction in shaft resistance due to a possible gap or reduction in horizontal pressure ends somewhere above the point were the deflection becomes negative. However, determining excat location of this point is beyond this study's scope, and assuming a deeper depth is conservative.

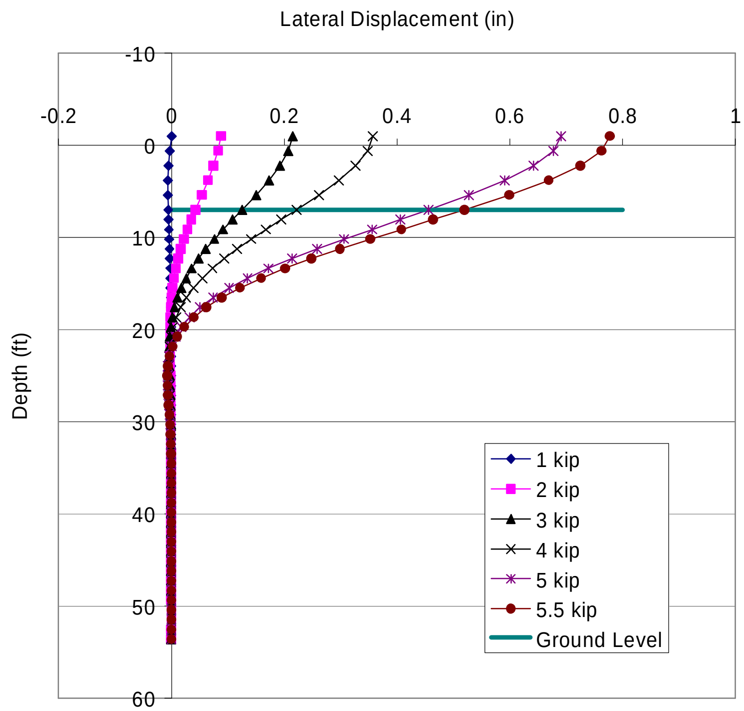


Figure 48. Robeson County Pile 1, Lateral Deflections under increasing lateral load

Once the gap depth was estimated, the shaft resistance model was changed such that the ultimate shear resistance between pile and soil for both the top and bottom of the affected length was set to effectively zero. The assumed fixed toe model for this bridge was replaced in MultiPier model to check the sensitivity of the results if the toe is allowed to displace freely. All other soil parameters and structural model parameters remained the same as entered for the analyses in Chapter 4.

A series of increasing axial loads was applied to each pile top, starting at a dead load of 25 kips per pile and increasing by an integer multipler until the model would no longer converge (indicating soil failure). This "pushdown" analysis was initially started without a modeled gap, and then increased to model a 9 ft long reduction due to a 0.04 inch lateral gap at the ground surface under a 2 kip lateral load. Finally, the worst case scenario was analyzed describing design load of 2 kips applied with a 0.5 inch gap and 13 feet of reduced strength. This simulates an overload followed by resumption of the design loads. The displacement under each axial load was recorded for each of the three loads. As shown in Figure 49, a difference in axial displacement does not become significant until five times the dead load is reached. At six times the dead load, the 13 foot strength reduction along the depth of the pile case does not converge.

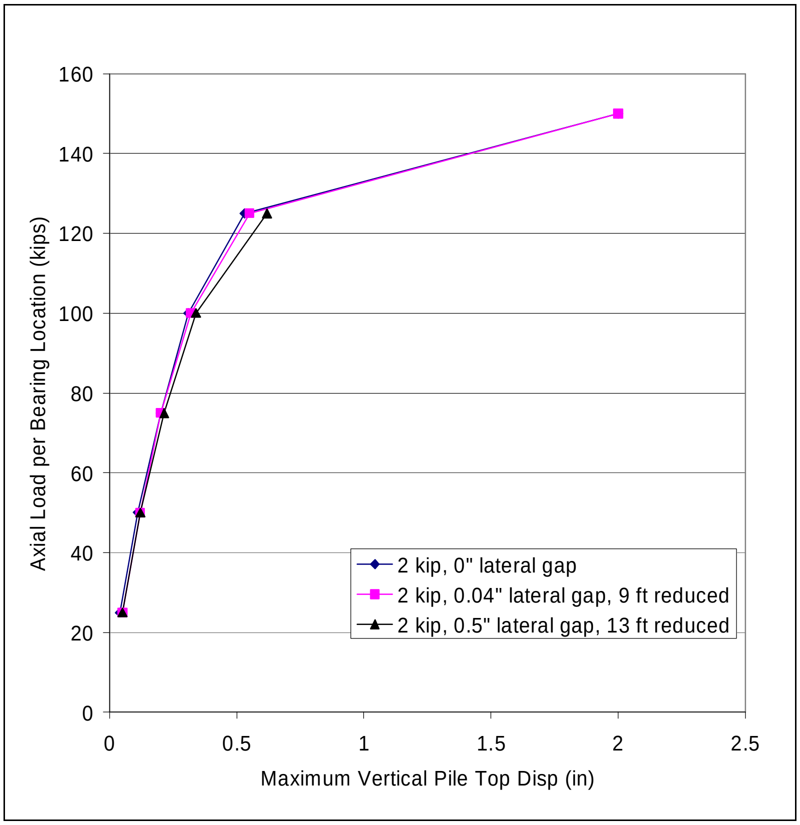


Figure 49. Robeson County, Maximum Axial Pile top Displacement under increasing axial loads with shaft resistance reductions

Piles in Clays with End Bearing

A study similar to that undertaken for the Robeson bridge piles was performed using the Northampton county bridge model. This bridge was constructed in a profile consisting predominantly of clay. However, there is a relatively dense, seven foot thick layer of sand at the ground surface, underlain by soft clays.

A series of lateral loads ranging from 0 to 75 kips was applied to the four bearing locations on this five pile bent. The axial load applied to each bearing location was 145 kips, or approximately the unfactored dead load. The laterally deflected shapes obtained for Pile 1 are shown in Figure 50.

From Figure 50, the gap developed at the ground surface and the distances from the ground surface to the point of sign change of the lateral deflection were noted. In this case, the gaps were as high as 1.2 inches, while the affected length along the pile shaft was as low as 9.5 feet at 0.04 inch gaps and as high as 21.2 feet at the larger 1 inch gaps.

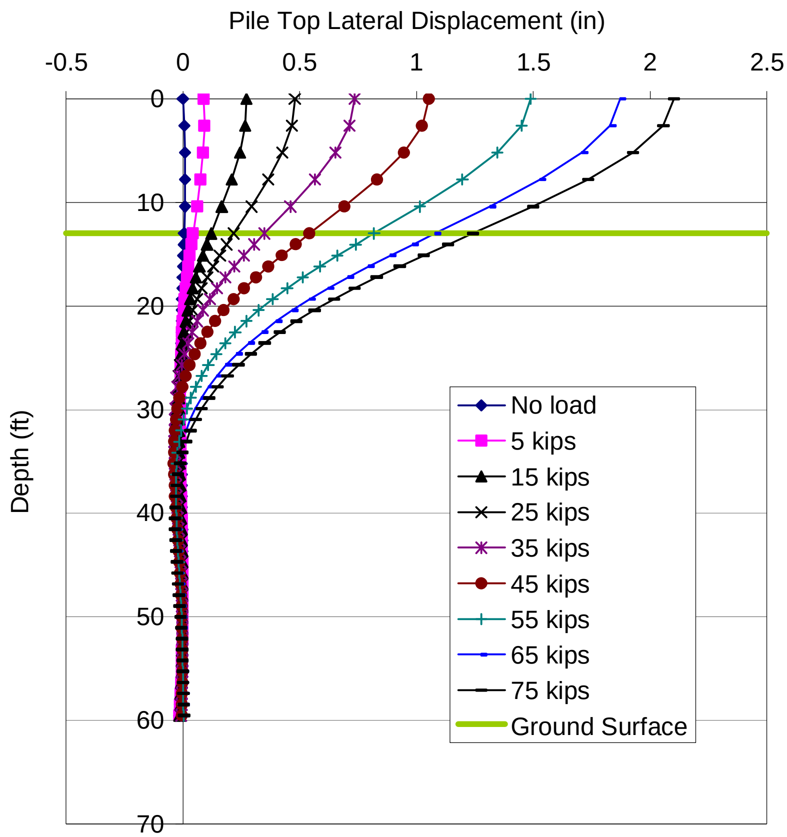


Figure 50. Northampton County, Pile 1, Lateral Deflections under increasing lateral load

MultiPier was re-run on the Northampton pile bent, this time applying a constant 5 kip lateral load and increasing the axial load at each bearing location. Axial load was increased until the model would no longer converge. A plot of axial load versus axial displacement at the top of Pile 1 is included in Figure 51. Similar to the Robeson county results, the reduction in upper shaft resistance does not have a large impact on axial displacement until, in this case, about seven times the dead load was applied.

Both the Northampton and the Robeson county bridges are similar in that they have high degree of toe resistance. Comparing similar lateral gaps of 0.5 inches, the Northampton model's reduced shaft resistance length was 14.8 feet, while for Robeson county 13 feet was affected. Thus, at similar gaps, similar depths were affected.

The Robeson and Northampton results seem to imply that, for piles whose capacity is generated mostly from end bearing, the loss of axial shaft resistance along the top part of the pile does not appear to have a large effect until either loads or displacements get very large. One would expect, based on these analyses, that a pile that develops its capacity purely from end bearing (i.e., a pile driven through very soft, weak soils to rock) will not experience reduction in axial capacity due to the opening of a gap.

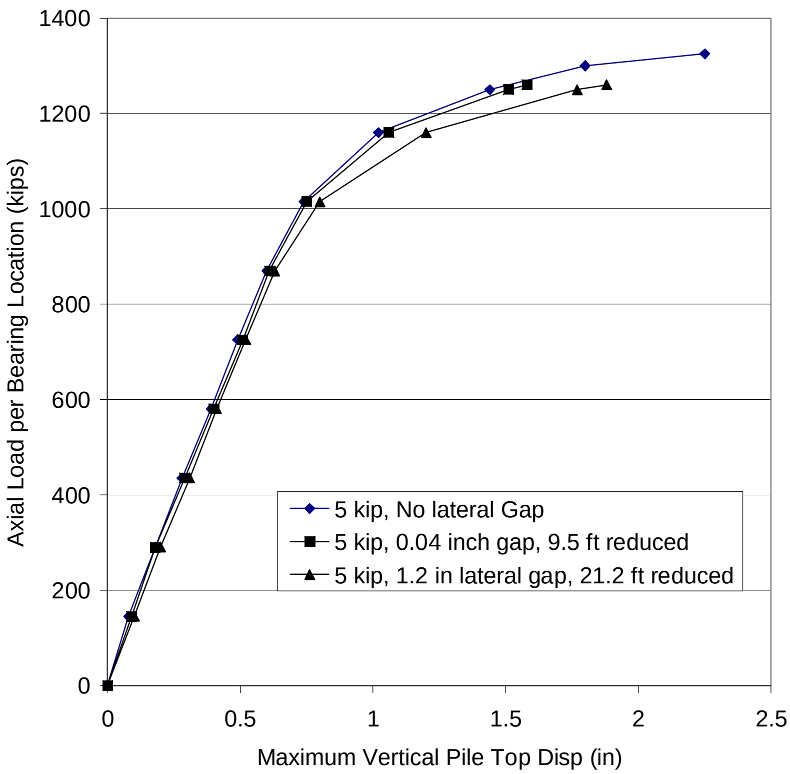


Figure 51. Northampton County, Maximum Axial Pile Top Displacement under increasing axial loads with shaft resistance reductions

Piles with friction (no end bearing)

The development of a gap may have the most effect on piles that develop most or all of their capacity from the skin resistance. This effect should be especially pronounced in relatively constant strength profiles (such as a uniform clay layer using the alpha method of pile capacity estimation). To that end, the Northampton bridge bent model was modified to model a uniform clay soil profile.

The Northampton soil profile was artificially modified such that upper sand layer and the lower sand layer were removed. The remaining soft clay profile in between was extended to the top of the ground surface past the bottom of the pile. Initially, the ultimate unit shear resistance was 400 psf along the entire length of the pile. The toe resistance was set to one kip to remove it from consideration.

Lateral loads were then applied to the model, this time ranging from 5 to 27 kips. The deflected shapes are shown in Figure 52. The gaps at the ground surface ranged from 0.07 inches at the five kip lateral load to one inch at the 27 kip lateral load. The reduced shaft resistance length ranged from 10.6 to 24.3 ft below the ground surface.

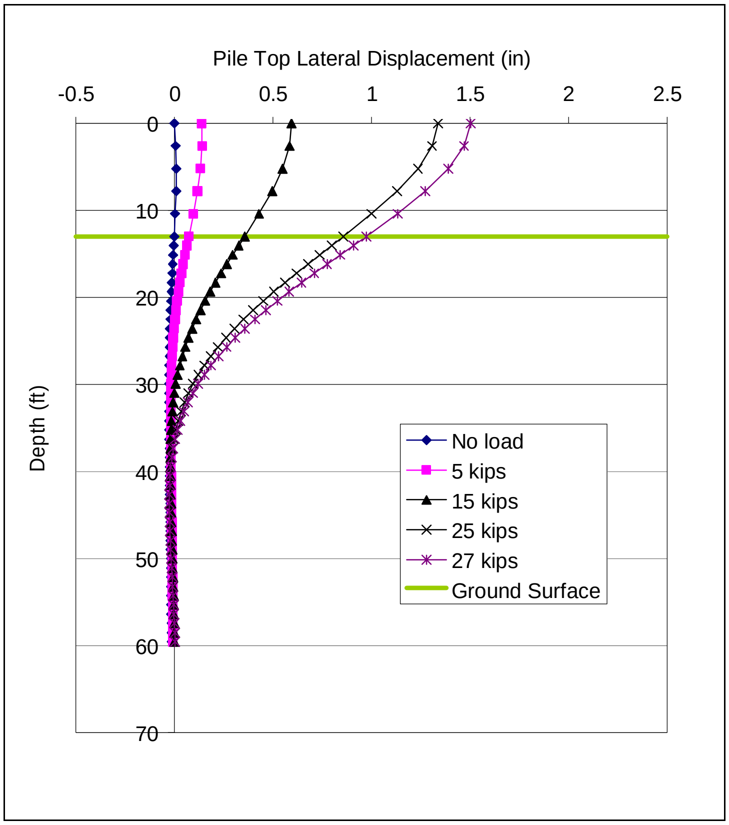


Figure 52. Deflected Shapes of Friction Pile Model

Once the reduced shaft resistance zone was determined, the model was axially loaded to failure. The results are shown in Figure 53. Unlike the previous two models, there is a noticeable difference as the lateral gap gets larger and larger. These results are not surprising, since removing half of the resistance contributing to the capacity from the shaft means removing half of the ultimate capacity.

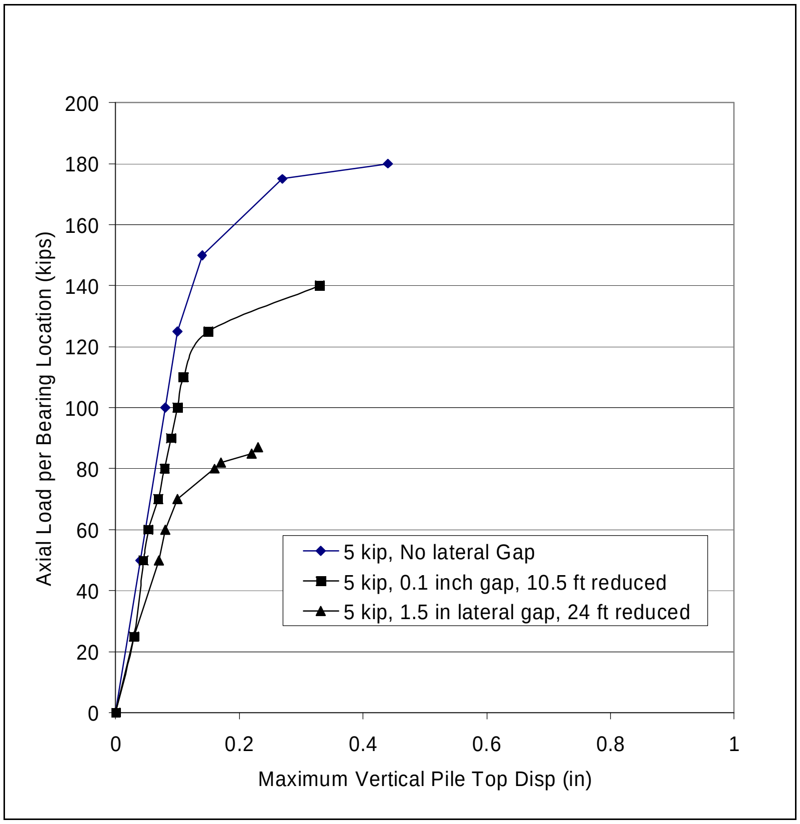


Figure 53. Axial Load-Displacement Curves of Friction Pile With and Without Gaps

Summary

In general, it appears that, for piles that have significant percentages of their ultimate capacity in end bearing, there is no particular limitation from an axial capacity perspective on the amount of lateral displacement that can be developed due to lateral loading. The gap at the surface that is developed affects only the upper part that contributes little to the overall axial capacity of an end-bearing type pile.

On the other hand, for piles whose resistance is due mainly to shaft resistance, gaps of 1 inch or more at the ground surface could have a large effect on the overall ultimate capacity of the pile. Accordingly, the use of friction piles to resist lateral loads, particularly in relatively soft soils, should be considered carefully.

Limit States for Bridges Under Lateral Loads

Various limit states can be defined for structural systems such as bridges. In fact, under a specific load, the response of the system will vary as well as the level of deformation or damage. For instance, under service loads, the deformation should not compromise functionality of the bridge, and the level of deformation of each member should be in the elastic range considering that no damage should occur under service loads. Consequently, under such level of deformation the structure will reach the ***Serviceability Limit State.***

That is not the case for other limit states such as *Damage Control* in which minor damage may appear in the system. The damage in this limit state should neither compromise the stability of the structure nor the integrity of the elements. It should be repairable and in some cases, as in earthquake mitigation measures, there might be certain secondary devices that are meant to be easily repairable and changeable. These devices dissipate energy through deformation and damage, and allow the system to have a better response under certain demands.

Finally, the structure should be able to tolerate the *Ultimate Limit State* or collapse in a safe manner considering that the security of the people and goods might be the priority. Limit states design allows the designer to go further in the analysis of a structural system up to the point of failure. Some considerations are taken into account such as the type of failure and the order in which the elements will start failing. In general, it is desired to have a ductile failure rather than a brittle one, and in many cases columns or piles are important elements that should not fail before less important elements (AASHTO, 2004).

It is important to notice that there are several levels of limit states that can be defined depending on the response of the structure under specific demands. A general description is shown in Figure 54, and it is important to notice that each limit state might be defined by a given factor such as yielding of steel, crushing of the concrete, and others. Moreover, limit states such as serviceability, damage control, and ultimate could be applied for each element separately, for a group of elements such as the substructure or the superstructure, or for the entire system.

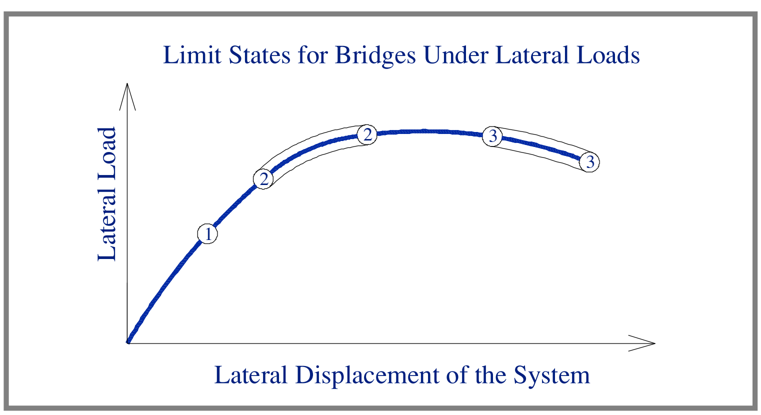


Figure 54. Idealized Limit States for a Bridge System

Figure 54 shows three zones identified in the load-displacement plot that can be generally described as follows:

1. *Serviceability*	Deformation limits for functionality purposes, usually elastic deformation
2. *Damage control*	Repairable minor damage in secondary elements.
3. *Ultimate*	Certain level of plastic deformation. Ductile failure of secondary elements.

Clearly, in case of bridges, the ultimate limit state should be avoided, as it would likely lead to the loss of the bridge as a useful structure. Some level of the damage control limit state may be acceptable in extreme events such as earthquake loading or hurricanes. Most, design, however, will work in the serviceability limit state as the desired criterion.

Limit States for Structural Members

Each element of a structural system such as columns or beams are subjected to axial loads, shear, and moments and limit states can be clearly defined. For instance, in the case of elements subjected to single bending, deflection can be obtained at different limit states. A formulation that considers both elastic behavior of a cantilever in single bending and development of a plastic hinge using the "Capacity Design Process for Flexural Ductility" section in Priestley et al. (1996). These concepts are generally used in earthquake engineering, but could also be applied to determine the serviceability limits state (or, up to the yield displacement) and ultimate limit states (up to the ultimate plastic displacement). These could then be compared to displacements due to applied loads for a particular member.

Limit States for Substructures

In the case of elements of the substructure of a bridge such as piles, some limit states might be defined in the following way:

o Serviceability	Defined level of deformation Maximum tolerable settlement Deformation beyond which repair is no longer feasible

o Ultimate	Severe plastic deformation affecting the stability of the system.

Limit States for the Superstructures

Limit states for the superstructure of a bridge my vary form bridge to bridge depending on configurations and type of structural system used. In general, limit states for the superstructure of the bridges analyzed in this project might be defined in the following way:

o Serviceability	Joint closure (may govern) Maximum tolerable deflection (functionality)
o Damage control	Repairable damage in some secondary elements such as expansion joints, slab or deck, connections, and others.
o Ultimate	Certain level of plastic deformation. Ductile failure of secondary elements.

Possible Limit States for Bridges Under Lateral Demands

It might be useful for design purposes to define lateral displacements limit states under lateral demands as follows:

1. **Serviceability** - Maximum Lateral displacement without damage in the system
   - Expansion joints closure (May govern - Possible to define)
   - Lateral deflection reaching a defined limit

2. **Damage control –** Some elements reaching yielding stresses and starting to develop non-linear behavior.
   - Minor damage in some connections
   - Minor damage in the superstructure
   - Undesirable effects in the substructure – displacements due to secondary effects.

3. **Ultimate**
   - Plastic deformation

- Excessive lateral deformation
- Failure of some connection (Expansion- joints)
- Damage in the superstructure and/or substructure

Considering the four bridges analyzed in this project, it seems that serviceability limit state under lateral demands may be governed by the closure of the expansion joints. In fact, the closure of the expansion joints in these cases may occur before the piles and other elements could reach their maximum capacity. Considering the functionality of the bridge, the closure of the expansion joints might represent a valid limit state in which the bridge has reached the maximum lateral displacement, and no damage has appeared in the bridge.

Limit States for Lateral Displacements Considering Superstructure Response

The following approach develops a serviceability limit state considering lateral force and displacement response in which the closure of the expansion joints may govern (see Figure 55). The objective is to model lateral superstructure displacement limits, and generalize the model for the most common cases.

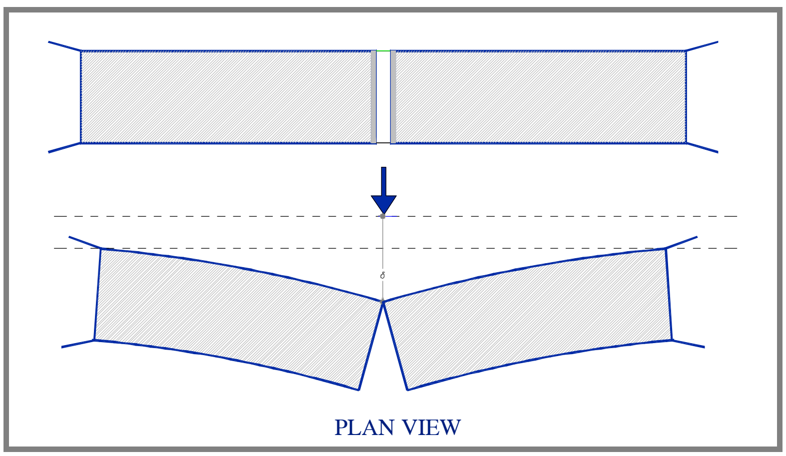


Figure 55. Closure of expansion joints: conceptual

Maximum Lateral Displacements

Various mathematical models are developed and proposed in this project in order to obtain maximum lateral displacement of a given bridge modeled as a whole beam with proper support idealization. The mathematical models take into account the various components of deformation as shown in Figure 56. These include lateral displacement due to deflection of the superstructure in the transverse direction, translation of the superstructure that is obtained considering the stiffness of supports in the transverse direction, and lateral displacement of the superstructure due to the rotation of the abutments considering its specific rotational stiffness when rotating in the horizontal plane. The variables that have been taken into account to develop the formulation in each case are:

- Stiffness of the pile group
- Abutments' stiffness
- Stiffness of the superstructure
- Boundary conditions
- Expansion joints location and width (DT – taken into account)
- Geometry of the superstructure (skewed angle, curved bridges, etc)

Proposed Simple Models: Serviceability Limits

The models have been developed considering the three components of deformation and relating them to the point in which the closure of the expansion joints may occur. The formulation takes into account only elastic behavior of the elements and the formulas are based on the principles of the simple beam theory. However, to get a better estimation of the response under applied load, it is important to use an appropriate moment of inertia of the superstructure cross section in the transverse direction, in order to account for the non-linear behavior of the section. In a simple manner, values of maximum lateral loads and corresponding maximum deformation can be estimated for the proposed serviceability limit state.

Future studies could optimize the procedure if translational and rotational stiffness of the substructural elements could be estimated in a more precise manner. It might be possible to classify different types of foundation elements such as pile groups or abutments and their stiffness. For practical purposes, the cracked moment of inertia of the most common cross-sections of superstructures could also be estimated and classified.

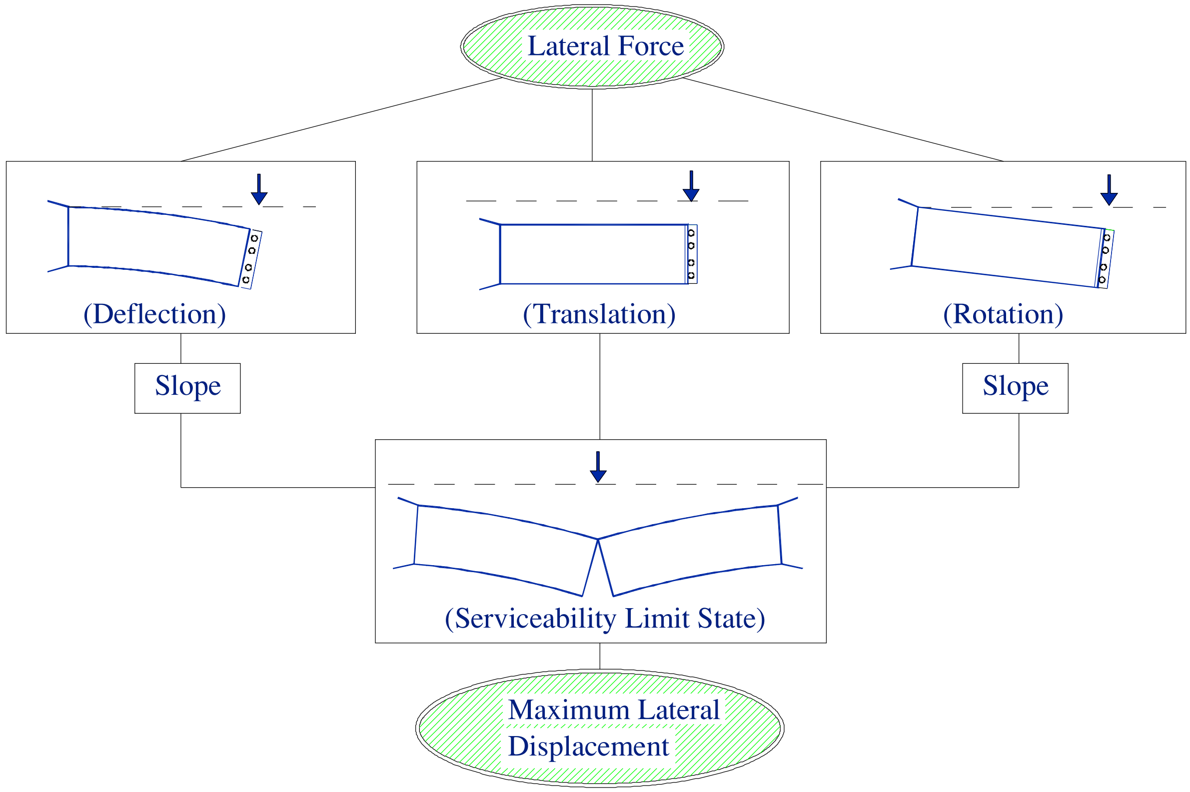


Figure 56. Breakdown of bridge deck deformations contributing to closure of joint

Defining Simple Models

Four preliminary models are proposed in Figures 57 to 60 to estimate the lateral force required to cause a lateral displacement of sufficient magnitude to close the expansion joints. Work on these models is ongoing, particularly due to uncertain rotational and translational stiffnesses at the abutment and at the connection between the superstructure and the interior substructure elements. Accordingly, these models are preliminary and their development will continue in the future.

Model #1: Specifications: (2 Spans – 1 Pile group in the middle – 2 Ex-Joints at the ends)

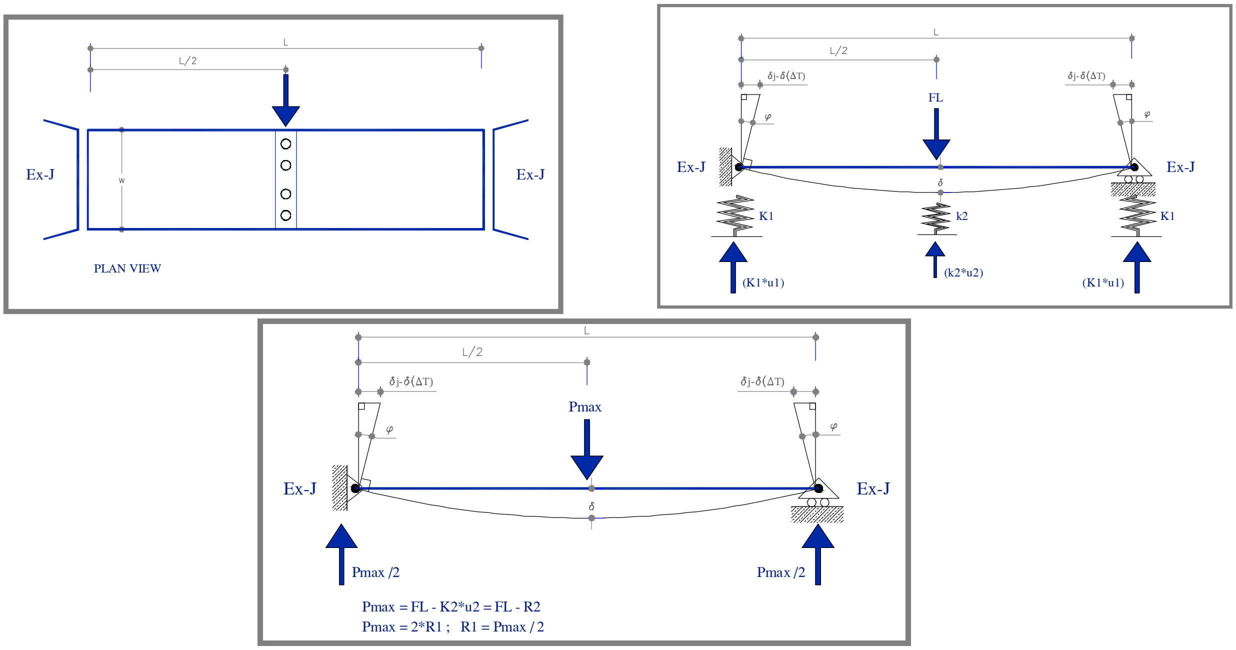


Figure 57. Model 1 Diagram, Fixed bent support, expansion supports at abutments

$$P\max = \frac{\left(dj - \dfrac{a(\Delta T)L}{2}\right)\cdot 16EI}{L^2\cdot(w/2)} \tag{6-5}$$

$$dtot = \frac{P\max\cdot L^3}{48EI} + \frac{P\max}{2\cdot K1} \tag{6-6}$$

$$FL = P\max + K2\cdot dtot \tag{6-7}$$

dtot = Lateral displacement limit (length units)

dj = Joint width (length units)

K1 = Abutment stiffness – (trans – dof) (force/length units)

K2 = Pile group stiffness (trans – dof) (force/length units)

Pmax = Force required to close expansion joint (force units)

w = Superstructure width (length units)

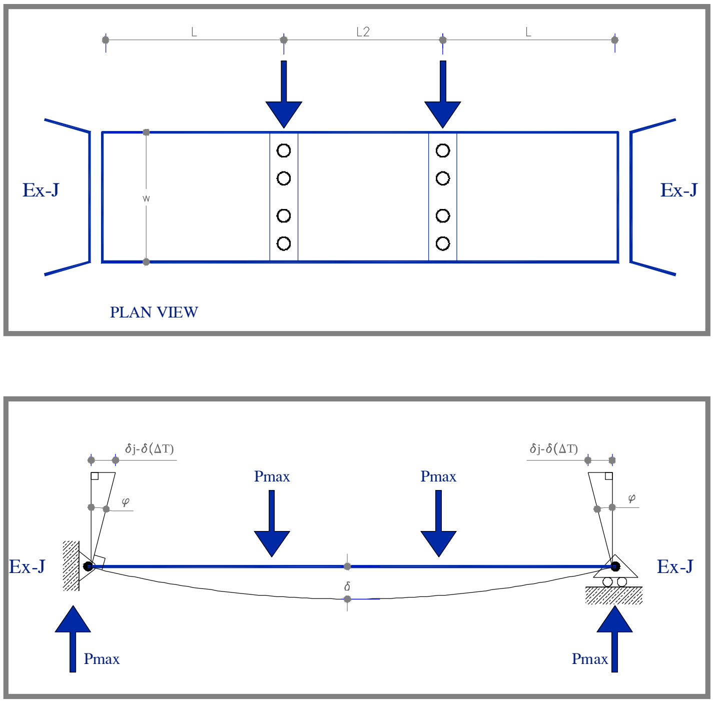
EI = Flexural stiffness of the superstructure (trans-dof) (force-length^2)

FL = Total Lateral load that will close the expansion joint and move the pile bent laterally (force units)

L = Span length (length units)

α = Coefficent of thermal expansion (1/Temperature)

Model #2: Specifications: (3 Spans – 2 Pile groups – 2 Ex-Joints at the ends)



$$dtot_{(@\,middle)} = \frac{23 \cdot P\max \cdot L^3}{24EI} + \frac{P\max}{K1} \qquad (6\text{-}8)$$

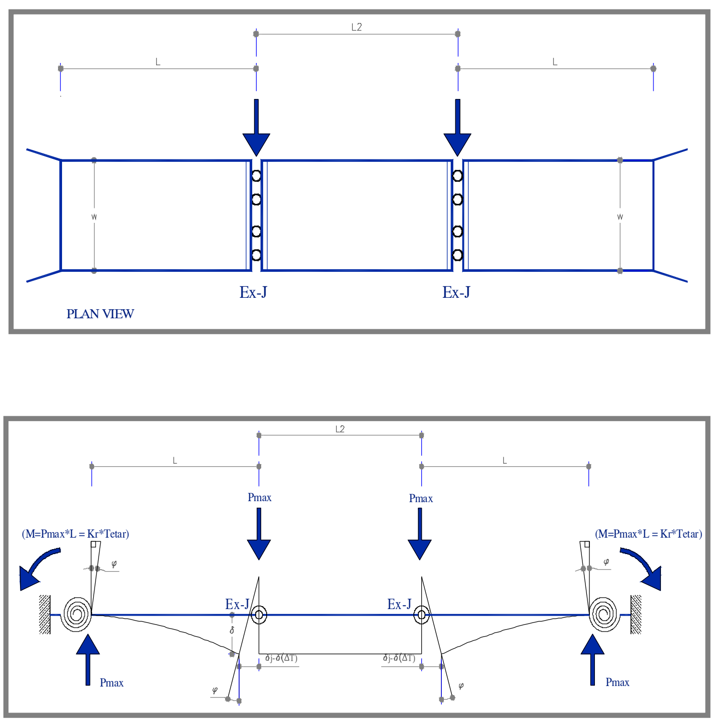
$$dtot_{(@\,Pile-Group)} = \frac{20 \cdot P\max \cdot L^3}{24EI} + \frac{P\max}{K1} \qquad (6\text{-}9)$$

$$P\max = \frac{\left(dj - a(\Delta T)\left(\frac{3L}{2}\right)\right) \cdot EI}{\frac{L^2 w}{2}} \qquad (6\text{-}10)$$

$$FL = P\max + K2 \cdot dtot_{(@\,Pile-Group)} \quad 11)$$

Figure 58. Model 2 Diagram: Fixed Bent supports, expansion abutments

Model #3: Specifications: (3 Spans – 2 Pile groups– 2 Ex-Joints at pile group location)



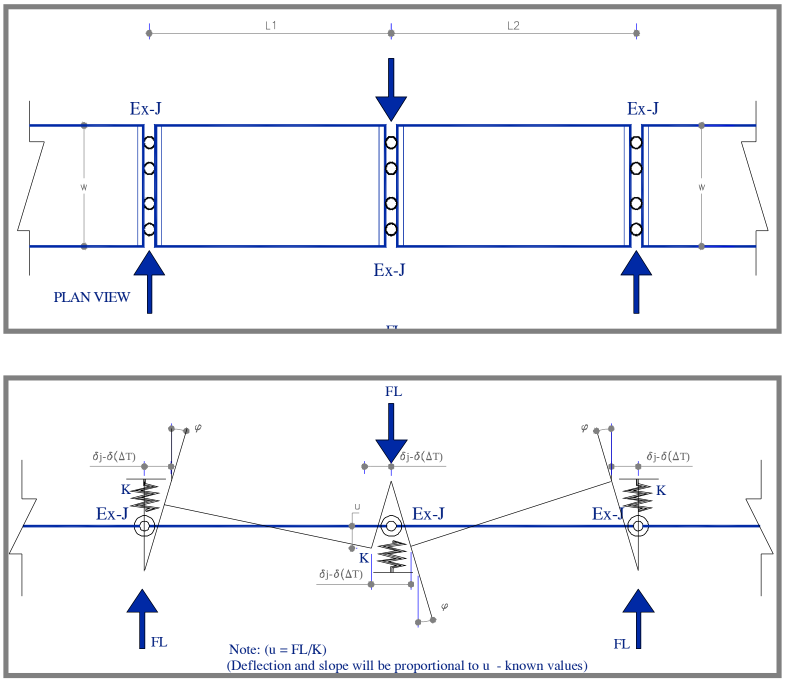
$$dtot = \frac{P\max \cdot L^3}{3EI} + \frac{P\max \cdot L^2}{Kr} + \frac{P\max}{K1} \cdot 12)$$

$$P\max = \frac{\left(dj - a(\Delta T)\left(L + \frac{L2}{2}\right)\right)}{\frac{w}{2}\left(\frac{L^2}{2EI} + \frac{L}{Kr}\right)} \qquad (6\text{-}13)$$

$$FL = P\max + K2 \cdot dtot \qquad (6\text{-}14)$$

Figure 59. Model 3 Diagram: Expansion bent supports, fixed end bents

Model #4: Specifications: (n# Spans – n# Pile groups – n#1 Ex-Joints at pile group locations)



$$dtot = \frac{(dj - a(\Delta T)\cdot L)\cdot L}{2w} \tag{6-15}$$

$$FL\max = \frac{(dj - a(\Delta T)\cdot L)\cdot L\cdot K}{2w} \tag{6-16}$$

$$dtot = FL\max/K \tag{6-17}$$

Figure 60. Model 4 Diagram, Multiple spans, expansion joints at each span

Detailed models

More detailed mathematical models can be derived considering all the possible variables involved in the bridge lateral response, various structural systems, different geometries, configurations, and all components contributing to the lateral stiffness of the superstructure. In order to use such models and formulation, it is important to accurately estimate the flexural stiffness of the superstructure in the transverse direction. The stiffness of the supports in the transverse direction must also be estimated.

For instance, lateral stiffness of the supports (pile bents or the abutments) can be estimated from their lateral force-deformation response. A MultiPier or SAP single bent pushover analysis can be used to determine the force-deformation curve, considering the interaction between the foundation and the soil. In the same manner, rotational stiffness of the foundation systems can be estimated and incorporated to the mathematical models. For example, the rotational stiffness of a pile bent subjected to some rotation of the cap beam could be estimated and derived using the lateral force displacement response of each pile, taking into account displacement at the top of each pile in relation to the angle of rotation.

Other models may consider the superstructure lateral stiffness component provided by connection between the superstructure and substructure, such as expansion joints made of neoprene bearing pads. While simple (or roller) supports are modeled as elements with no stiffness in one direction, such as the direction of traffic, they may still contribute to the superstructure's lateral stiffness in the transverse direction. It is believed this additional stiffness at the connection level helps reduce displacements due to applied lateral loads.

Numerical Example

As an example, a two span bridge with expansion joints will be considered as shown in Figure 61. The expansion joints will have neoprene bearing pads under the girders of the superstructure. Resultant lateral forces (from wind, stream flow, impact or other sources) applied to the middle supports cause a lateral displacement, consequently some relative rotation between adjacent decks, and eventually closure of the expansion joints. A detailed model will consider the contribution to lateral stiffness of the superstructure provided by the connection through the neoprene bearing pads. The shear strain of the pads will depend on the displacement at each pad location which will be proportional to the rotation and the distance from each pad to the center of rotation. Finally, the rotational stiffness of the support connection up to the serviceability limit state (or closure of the expansion joint), will depend on the shear resistance of the bearing pads to the displacement in the direction of traffic at each location. Beyond the expansion joints closure, the stiffness of the connection will increase significantly due to the added stiffness and yield strength of each deck and in this case, a new model would be needed to predict behavior beyond such serviceability limit state.

For this specific example, the superstructure lateral stiffness component due to the rotational stiffness of the connection at the supports will be considered. The mathematical model for this case was derived as shown in Figure 61, and equations 6-18, 6-19, and 6-20. Model #5 includes a rotational stiffness variable for the expansion joint connections. It also considers the stiffness of the pile bents. In this specific case, the flexural stiffness of the deck is assumed to be orders of magnitude greater than the other components' stiffnesses approaching serviceability limit state. So in this formulation the deck was assumed to be rigid. Finally, for this specific example no translation of the abutments is considered.

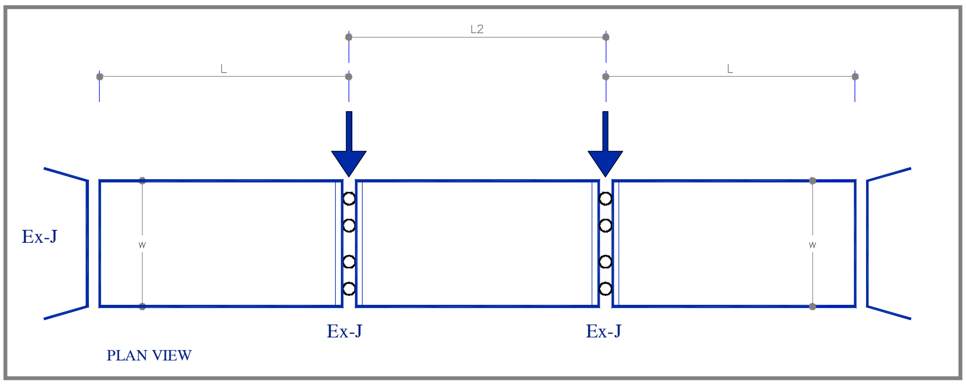
Model #5:

Specifications: (3 Spans – 2 Pile groups– 4 Ex-Joints at pile group location)

$$dtot = \frac{P\max \cdot L^2}{2Kr} \tag{6-18}$$

$$P\max = \frac{\left(dj - a(\Delta T)\left(\frac{L+L2}{2}\right)\right)}{\frac{w}{2}\left(\frac{L}{2Kr}\right)} \tag{6-19}$$

$$FL = P\max + K2 \cdot dtot \tag{6-20}$$



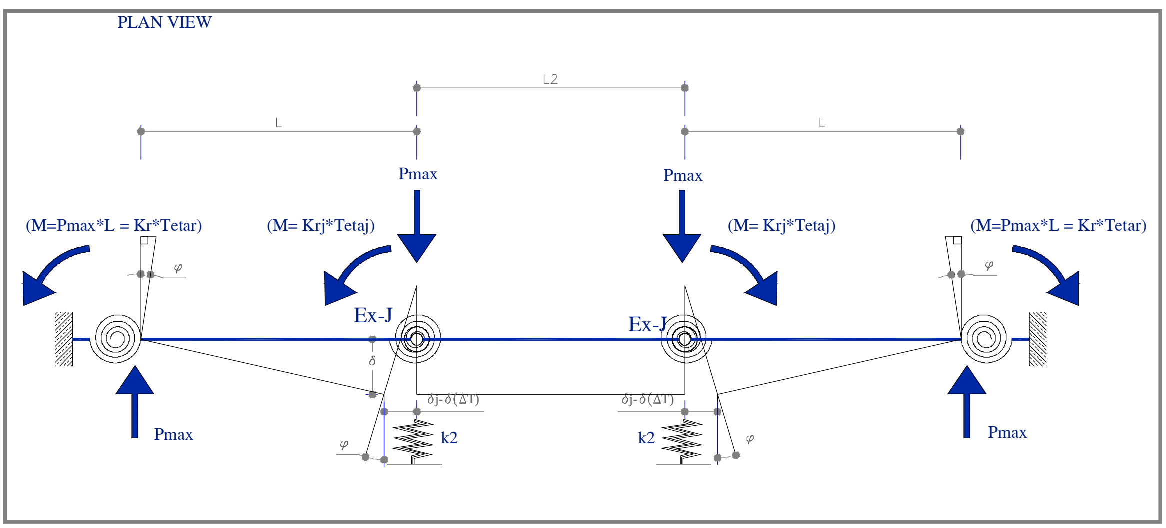


Figure 61. Model #5 – Includes Rotational Stiffness of the Connections

Pile Bent Stiffness and Rotational Stiffness of the Expansion Joint Connections

In the formulation shown for this specific case, Figure 61, and equations 6-18, 6-19, and 6-20, there are two input variables that require some detailed analysis. One is the required pile bent lateral stiffness (K2) that can be estimated from a lateral force-deformation response of the pile bent, where the stiffness is defined by the level of deformation predicted by the analysis, a value that may take some iteration in equations 6-18 to 6-20. This analysis was done in MultiPier for a single pile bent. The second variable is the rotational stiffness of the connections at the location of supports. In this specific case, the rotational stiffness of the connection from zero degrees up to a maximum angle determined by the expansion joints closure will be provided by each simple support's resistance to relative longitudinal displacements between superstructure and substructure (traffic direction). As shown in Figure 62, the displacement in the direction of traffic, at each simple support under each girder location is proportional to the rotation of the superstructure's deck. If the simple supports at the expansion joints have been designed with neoprene bearing pads, and if the stiffness of the bearing pads is considered, the rotational stiffness of the connection could be derived as shown in Figure 62, and equation (6-21).

The rotational stiffness of the connection for this specific example will be the sum of each different reaction at each pad location times the distance to the center of rotation divided by the total angle of rotation. If the shear force versus the lateral displacement response of the neoprene bearing pads is known, and the geometry of the bridge is defined in terms of the number and location of the pads, the moment versus rotation response of the connection could be derived, and hence the rotational stiffness is defined.

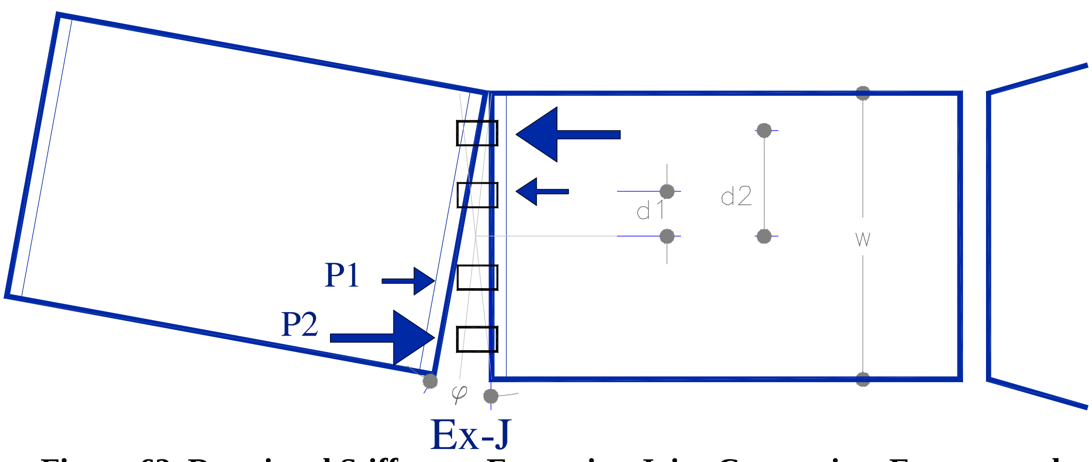


Figure 62. Rotational Stiffness – Expansion Joint Connection, Exaggerated

In general terms, equation (6-21) defines the rotational stiffness, Kr, for expansion joint connections, where “P_i” is the force at each bearing pad location as shown in Figure 59, “d_i” is the distance from the centroid of the bearing pad to the center of rotation of the superstructure, and “q” is the angle of relative rotation between two adjacent elements of the superstructure connected by the expansion joint in radians. If desired, a resisting moment at the connection at a given relative rotation could be estimated by multiplying the rotational stiffness, Kr, by the relative rotation in radians.

$$Kr = \frac{\sum_{i=1}^{n} P_i \cdot d_i}{q} \tag{6-21}$$

$$\mathbf{P_i = K(pad)_i * D_i}$$

$$D_i = f(q) = q * d_i \text{ ; (determined by geometry)}$$

The rotational stiffness can also be determined from the slope of a moment versus relative rotation response graph of the expansion joint connection, as shown in Figure 61. The slope “Kr” may not be constant unless the response is linear. The moment versus relative rotation response of the connection is a function of the bearing pads stiffness and its linearity depends directly on the shape of the shear force versus shear strain response of the bearing pads. From equation (6-21), equation (6-22) has been derived to simplify the calculation of the rotational stiffness without considering a specific angle, and assuming a linear behavior of the bearing pads material in a relative rotation range of the connection from zero to the expansion joint closure. In Equation (6-22), “K_{pi}” is the stiffness of bearing pad “i” for shear forces and displacements in the direction of traffic (as defined in Equation 6-23).

$$Kr = \sum_{i=1}^{n} K_{pi} \cdot d_i^2 \tag{6-22}$$

Rotational Stiffness of an Expansion Joint Connection – Numerical Example

For the following example, the rotational stiffness of an expansion joint connection will be estimated considering a simplified geometry of a fictitious superstructure cross section as shown in Figure 63.

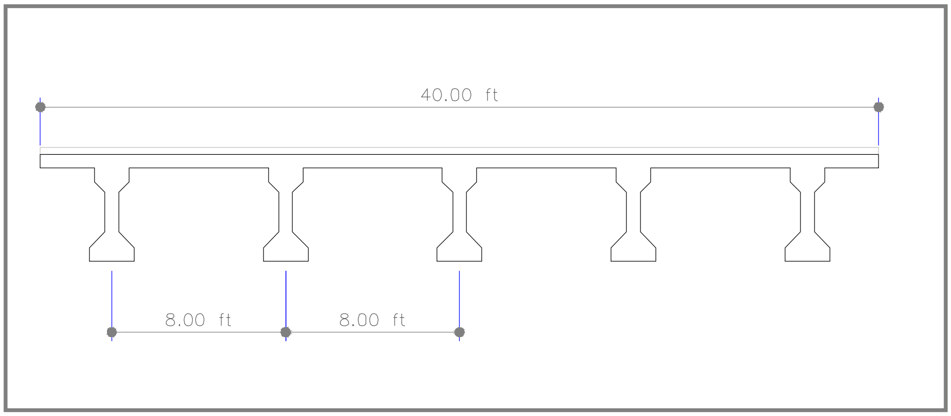


Figure 63. Superstructure Cross Section – Numerical Example

In order to illustrate the procedure, Figure 64 shows conceptual graphs of lateral force versus displacement response of the bearing pads, and from there, the desired moment versus rotation of the expansion joint connection. From experimental studies (such as Muscarella and Yura, 1995), lateral force versus displacement response of specific bearing pads can be obtained. The lateral force at each bearing pad location is obtained from the force-displacement response considering the displacement at each location for certain rotation. From there, the moment versus rotation response of the connection can also be obtained, and the rotational stiffness of the expansion joint connection is defined by the slope of the curve.

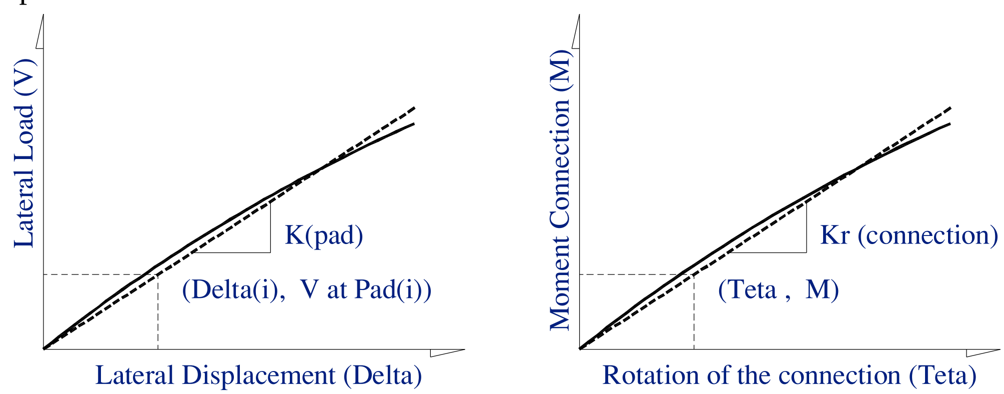


Figure 64. Conceptual Elastomeric Bearing Pad Response Curves

For the numeric example, the response of the moment versus rotation response of the expansion joint connections and the response of the five bearing pads are assumed to be linear in the desired strain/rotation range. In addition, the hardness of the bearing pads is assumed to be between 50 to 55 Durometer, the shear modulus, G, is 90 psi, the area of the bearing pads, A, is 200 in^2 and the thickness of the bearing pads, t, = 2 in. Finally, the stiffness of the bearing pads, assumed to be linear under service demands, is estimated using equation (6-23) and for this specific example has a value of Kp = 9.00 kips/in.

$$K_p = \frac{G \cdot A}{t} \tag{6-23}$$

Once the stiffness of the bearing pads is estimated, the geometry of the cross section is known, and the location of each bearing pad is defined, it is possible to determine the rotational stiffness of the expansion joint connections. For this specific example, considering the bearing pads are 8 feet apart on the cap beam, and the geometry of the superstructure cross section as shown in Figure 63, the rotational stiffness of the expansion joint connection is estimated using equation (6-22). For this specific numerical example it has a value, Kr , of 69,120.00 kip-ft/rad.

Finally, using the model #5 formulations proposed in Figure 62, equations (6-18 to 6-20), the lateral displacement limit and lateral force can be determined for serviceability limit state. In order to simplify and illustrate the calculations of this example, the joint width after an average thermal expansion of the superstructure (the numerator in Equation 6-19) will be considered to be one inch. The width of the superstructure for this example is considered to be w = 40 ft, and the length of the spans L = L2 = 80ft. Using equation (6-19) and (6-18), the maximum deflection of the superstructure before any damage may occur in the superstructure, or at a point in which the expansion joints are closed (serviceability limit state) is δtot = 4 inches.

Using equation (6-20) and assuming the stiffness of the pile bent group at each support location to be K2 = 70 kip/in based on a pushover analysis of the Halifax County Bridge Bent in MultiPier, taken just as a reference value for this example, considering the difference in the configuration and geometry between Halifax County Bridge and this example bridge, the maximum lateral force to reach the serviceability limit state of the superstructure at the closure of the expansion joints can be estimated. However, for the Halifax County bridge, MultiPier predicts failure of the substructure (demand/capacity ratio more than 1.0) for lateral deformations of 4 inches. Thus, it is clear that the maximum lateral load that will close the expansion joints should be estimated taking into account other limit states and ultimate capacities of elements such as substructure, connections, and others.

Given the full nonlinear force-deformation response of the pile bents (that is, including plastic deformations of the piles or bent cap) in the transverse direction, damage levels in the piles could be assessed for a given lateral displacement of the superstructure. For instance for the serviceability limit state of the superstructure (expansion joints closure), the pile bents may have already failed, reaching the substructure component's ultimate limit state, something not desired in a bridge design. Alternately, the pile bents may not have failed, but the damage could be very significant and may preclude repair. Finally, the pile bents may not have reached their serviceability limit state. Consequently as is often desired in a bridge design, damage may occur first in the superstructure.

Structural Limit States: Bent Displacement Required to Cause Elastic Yield in Piles

For each of the four bridge case studies, analyses were performed to determine at what displacements (of the pile tops/bent cap) elastic yield strains develop. To accomplish this, the SAP nonlinear models were run for each of the four bridges. As discussed in Chapters 4 and 5, the piles were modeled in both programs with nonlinear soil and pile elements, while the concrete bent caps were modeled with elastic elements. Pushover analyses were run for each of the four bents, with the axial dead load case applied to the bearing locations. For the pushover, an increasing lateral load in the transverse or longitudinal direction was applied to each bearing location, as well. For each load case, the SAP results were checked to determine the transverse or longitudinal displacement at which elastic yield was reached.

The elastic yield of each of the four pile sections was determined by moment curvature analysis using the Response2000 program (Bentz, 2001). Based on this analysis, the applied moment required for development of concrete crushing or steel yielding was selected for each of the four pile sections. Figures 65 and 66 show example moment-curvature analyses from the Robeson County Bridge H-Pile section and the Halifax County Bridge square prestressed concrete pile section. The remaining moment curvature analyses are available in the Electronic Appendix. For entry into the SAP program, a bilinear model was assumed for all four analyses.

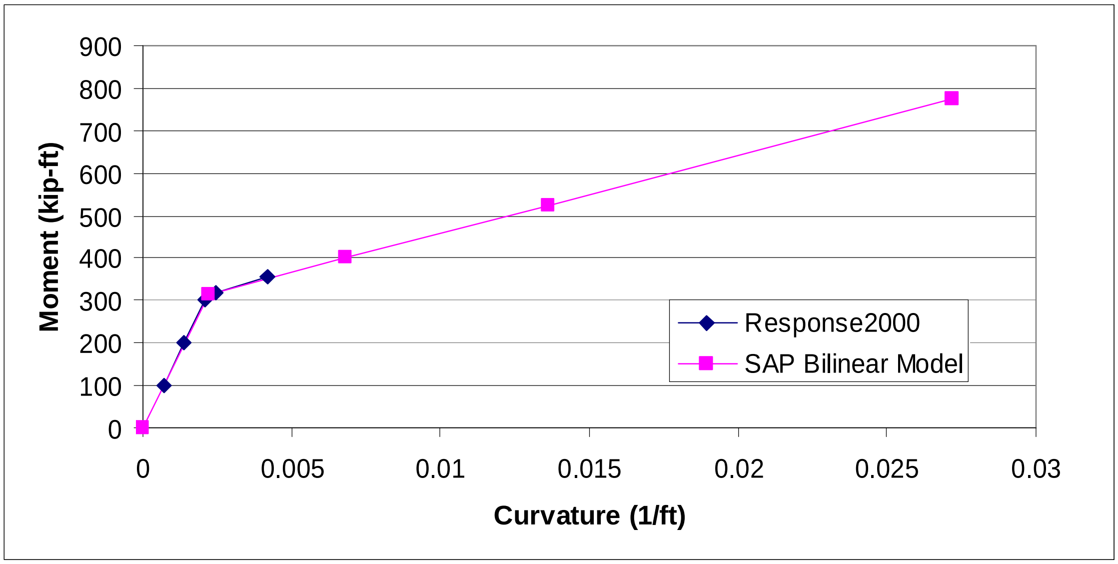


Figure 65. Robeson County HP14x73 Moment Curvature Response--Strong Axis

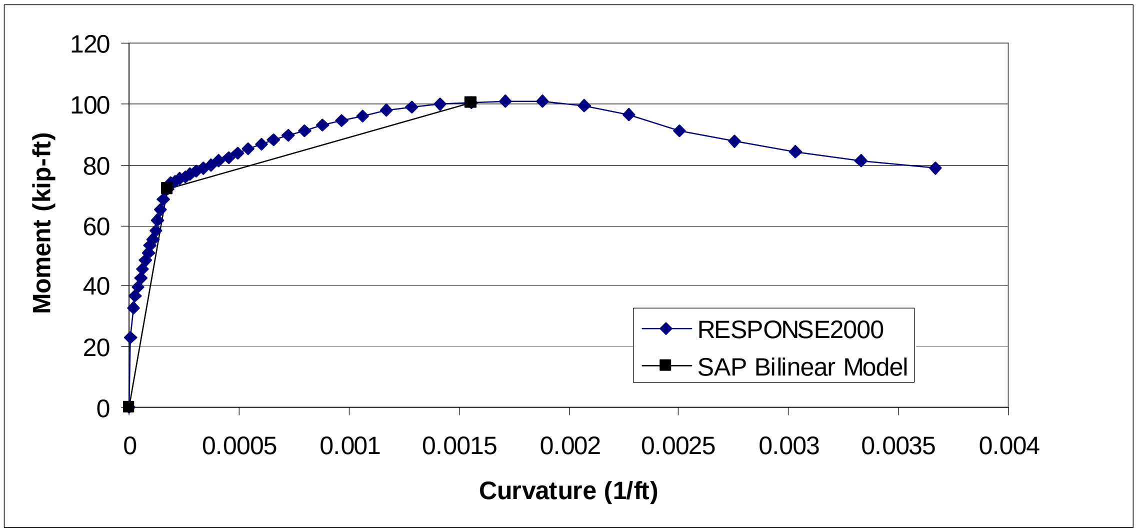


Figure 66. Halifax County 18" Square Prestressed Concrete Moment Curvature Response

For Figures 65 and 66, the moment required for elastic yield is roughly defined as the location of an inflection point in the moment-curvature analysis. For the Robeson County H-Pile, the yield point in the strong axis is at a moment of approximately 300 kip-ft. The Halifax County moment curvature analysis is not as clearly bilinear, due to the nature of concrete. In this case, the inflection point for “elastic” yield occurs around a moment of 75 kip-ft. Clearly, the bilinear approximation used in SAP more closely matches the Response2000 results for the H-Pile than the concrete pile. For the purposes of these analyses, the approximation should suffice, however.

Once the moments required to reach yield for a particular axial force were determined, the pushover analyses were run. Figures 67 and 68 show the overall transverse load-displacement results from SAP for the Robeson and Halifax bents, respectively. Similar to the moment-curvature results, points of inflection can be noted that show where the piles are being pushed beyond their elastic limits. In Figure 67, this inflection point occurs at a total transverse load of approximately 175 kips, while in Figure 68 this point occurs at a total transverse load of approximately 75 kips.

In Figure and Figure and in the Tables 2 and 3, the total transverse load applied to the bent is the product of the total number of bearing locations and the transverse lateral load applied to each bearing location on a pile bent. During pushover analyses, this load is incrementally increased. The transverse displacement shown is the maximum displacement displayed for the pile tops in each bent. While there was some difference in displacement from pile top to pile top in a particular bent, these differences were negligible compared to the overall magnitude of the displacement.

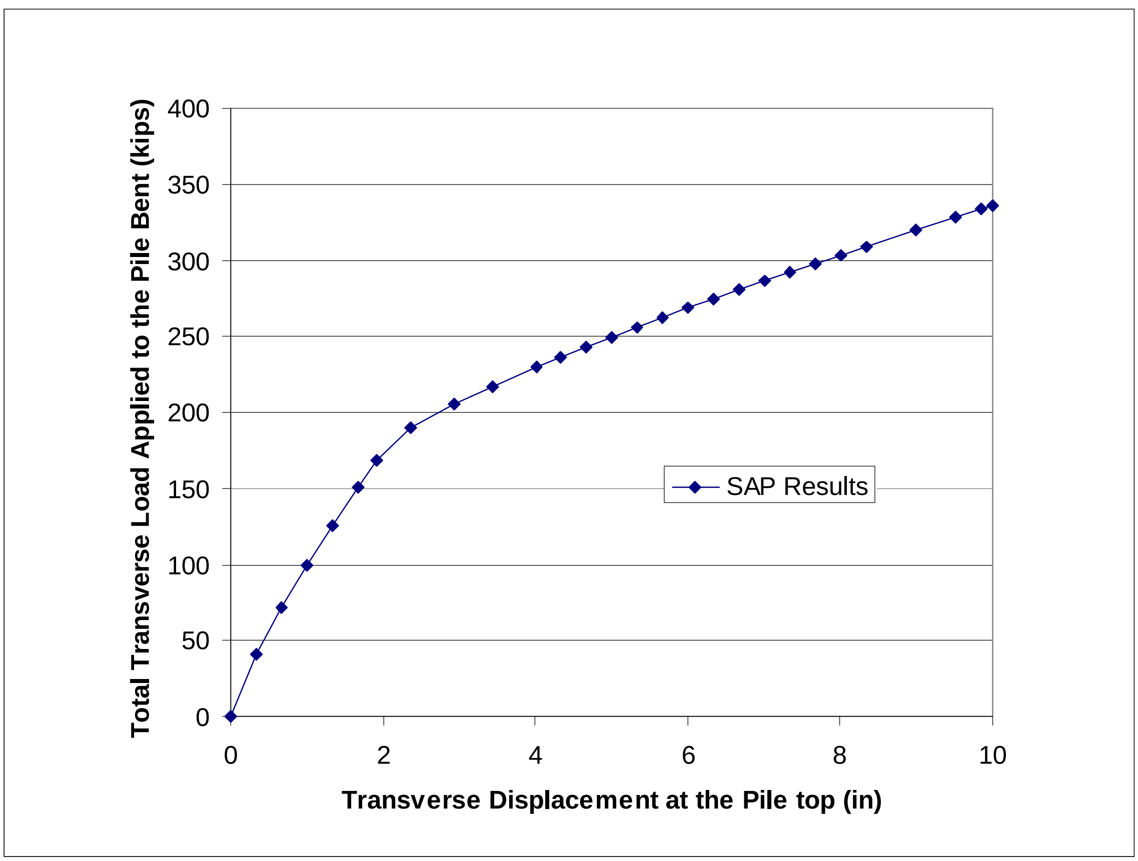


Figure 67. Simulated Bent Load-Displacement Curve from SAP: Robeson County

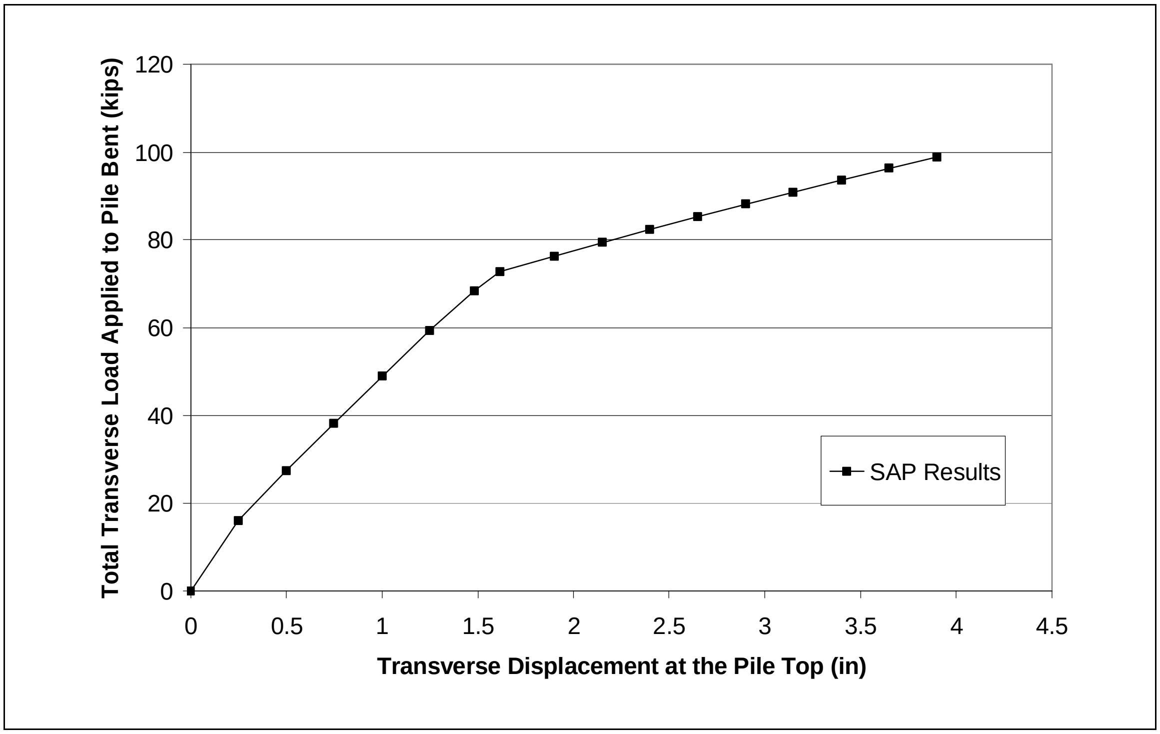


Figure 68. Simulated Bent Load Displacement Curve from SAP: Halifax County

Table 30 summarizes the results from the pushover analyses in the transverse direction. The total transverse load displayed in Table 1 is the load at which elastic yield of the pile sections occurred. For comparison, the total transverse load applied in AASHTO Group II loading, with the wind direction parallel to the transverse direction of the bent, are also shown.

As shown in Table 30, elastic yielding occurs at slightly more than two inches of transverse displacement. However, based on the work presented in Chapter 5, it should be stressed that the transverse direction appears to be best modeled as a fixed head condition in single pile analyses. For the DOT's existing LPILE runs, a free head is generally assumed.

Another set of pushover analyses were run for loads in the longitudinal direction only, using the same methods as for the transverse analyses. These results are summarized in Table 31. Assuming the pile bent is free standing, the SAP results show the piles are not expected to yield until displacements of five or more inches. Again, the total longitudinal load applied in the pushover analyses are generally many times the load actually applied in the original analysis. In the longitudinal cases, however, the concrete pile bents from

Halifax and Washington counties appear to have smaller differences between the pushover loads for yield and the applied loads (factors of 2 and 4, respectively). In these cases, it would appear the soils are entering a highly nonlinear range prior to elastic yielding of the piles.

Table 30. SAP transverse pushover analysis results for elastic yield in piles.

As-Built Bridge	SAP Transverse Displacement at Yield (in)	SAP Total Transverse Load on Bent at Yield (kips)	As Designed Total Group II-1 Factored Transverse Load (kips)
Robeson	2.10	178	17
Northampton	2.14	289	52
Halifax	2.07	73	16
Washington	2.60	168	50

Table 31. SAP longitudinal pushover analysis results for elastic yield in piles.

As-Built Bridge	SAP Longitudinal Displacement at Yield (in)	SAP Total Longitudinal Load at Yield (kips)	As Designed Total Group II-5 Factored Long, Load (kips)
Robeson	5.01	232	14
Northampton	10.3	220	31
Halifax	9.50	38	18
Washington	9.20	58	13

Table 31 should be interpreted considering the other potential points of failure in the bridge. While SAP says a free standing pile bent can move 5 to 10 inches before onset of elastic yield in the piles, it currently says nothing about the state of the expansion joings in the bridge, the elastomeric bearing pads or the connection between the cap beam or bridge deck. Some of these considerations are roughly considered previously in this chapter; others will be covered in future research.

To check existing displacement limit states, Liu et al.'s (2001) determination based on 2% of the free column heights was used. These results are summarized in Table 32. The results of this simple analysis compared relatively favorably in the transverse direction for three of the four bridges, but seem to slightly underpredict the required displacement in the concrete piles of Washington Counties. In the longitudinal direction, the free column height method drastically under predicts the yield point of a free standing pile bent.

Table 32. Comparing SAP results to simple limit state by Liu et al. (2001)

Bridge	2% of Free Column Height (in)	SAP Transverse Displacement at Pile Yield (in)	SAP Longitudinal Displacement at Pile Yield (in)
Robeson	1.92	2.10	5.01
Northampton	1.92	2.14	10.3
Halifax	1.92	2.07	9.50
Washington	1.58	2.60	9.20

The results of the pushover analyses can be considered in light of the NCDOT's "one inch" lateral displacement limit imposed during the lateral pile analyses, keeping in mind that the scope of this study is limited by the small number (4) of bridges analyzed. With this limitation in mind, it would appear that, for transversely loaded bridge bents, lateral deflection of the piles can be increased to two inches before yielding occurs. For a free standing bent loaded longitudinally, two inches is a conservative value for the four bridges studied. Similarly, the simple lateral displacement limit of 2% of free column height suggested by Liu et al. (2001) was conservative for the four bridges studied in both the transverse and lateral direction. For more general application, however, further cases should be examined that consider a typical range of soil types, pile and free column height conditions in North Carolina.

Summary

This chapter highlights the necessity of determining limit states for each element of the bridge and the necessity to relate them simultaneously such that an efficient design assures the order in which damage may occur in each element of the bridge. Even though further research is necessary to accurately estimate the lateral force-deformation response of an entire bridge for demands such as earthquakes and wind loads, this chapter proposes models to estimate serviceability limit states for various bridge superstructures based on closure of expansion joints. These models should not be used without taking into account the limits states of other elements simultaneously in order for the designer to get a better understanding of the behavior of the entire system.

CHAPTER 7: SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

Work in this study included a review of the NCDOT's current design practices for pile bents and provided two alternative approaches for performing design and analysis. Two software suites were utilized for this purpose—SAP2000 and MultiPier. The SAP and MultiPier analysis results were compared to one another using a variety of analyses to verify the methods of both programs. Their results were compared to those from Georgia Pier, the program traditionally used by the NCDOT for design of pile bents.

Four bridges representing three different pile types and four different superstructures were selected to evaluate the current design practice. First, these bridges were modeled by nonlinear analysis of a standalone pile bent in both SAP and MultiPier. Once the nonlinear models were completed, possible cost savings were explored by reducing the number or type of piles in the MultiPier models. The linear frame model (Georgia Pier) was evaluated against the nonlinear models, and its limitations were indicated. An improved frame model was proposed and evaluated using the four bridge case studies. Finally, an investigation of limit states was performed and preliminary models to quantify the substructure-superstructure interactions were developed and presented.

Summary

The current NCDOT design practice calls the Geotechnical Unit to run preliminary pile design calculations, including determining a point of fixity from LPILE and other analyses based on a set of assumed loads. The pile size, pile length and point of fixity are then transmitted to the Structures Unit, where an equivalent frame model is input into the Georgia Pier program. Georgia Pier and the NCBDS subroutines are used to design the bent as a free-standing elastic frame when subjected to various AASHTO load combinations. The programs outputs include reinforcement magnitude and dimensions for the bent cap beam as well as service loads to finalize pile design. The following comments on the current design approach are offered:

1. Georgia Pier does not calculate displacements of the pile bents. Such data would be convenient for directly comparing to displacement limit state values although a literature review yielded few existing studies on limits to bridge movements..

2. The approach of using LPILE to determine POF with preliminary loads should yield similar moment and displacement results when placed in a frame subjected

to AASHTO loading; refinement of the point of fixity determination or the LPILE loads may be needed.

3. Pile data input into Georgia Pier assumes all piles are vertical (that is, the function for analyzing piles installed on a batter reportedly does not work properly.)

4. For H-Pile sections, the pile's strong axis maybe at times rotated as shown in the plan view in Figure 1 (recreated below as Figure 69). Georgia Pier does not model piles with two different strong axis orientations in the same bent.

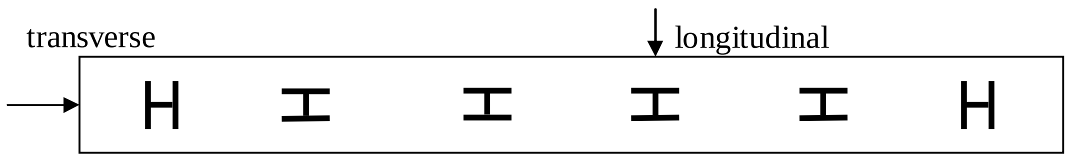


Figure 69. Plan View: H-Pile Bent with Rotated Pile Axes

5. For concrete pile sections, Georgia Pier requires that the concrete in the bent cap has the same elastic modulus as the concrete in the piles, which forces the designer to input equivalent sections if different moduli are used for reinforced concrete bent cap and prestressed concrete piles.

6. Georgia Pier does not model soil, and therefore does not capture pile group effects, which tend to reduce stiffness compared to assuming a single laterally loaded pile representing group behavior.

7. The k-values input in Georgia Pier are taken from AASHTO tables meant for unbraced steel columns in frames. These determine the steel's buckling behavior, and are currently used to magnify moments due to second order effects and to calculate a structural axial capacity of the pile. These k-values do not account for soil-pile interaction. In the case of nonlinear analyses which takes into account second order moment effects (as many lateral pile analysis programs, including LPILE, can do), this k-factor would be redundant.

8. The Georgia Pier program uses the method of moment distribution to obtain moments, shear, and axial forces, but does not consider any translation of the joints due to movement of the piles. Thus, comparing results from Georgia Pier to the "real" behavior of the structure allowing vertical translation of joints may yield significant differences in response.

3-D Nonlinear Models

In this study, SAP and MultiPier were used. SAP was implemented for comparison purposes and as an independent verification tool of Multipier results. MultiPier is specific to bridge applications and has built-in soil models and other pre-packaged features. If, however, MultiPier is used as design tool, some preprocessing would be needed to determine live loads, and to efficiently enter all load cases into the MultiPier program. Postprocessors are also required for design of bent cap section reinforcement.

The soil models in MultiPier follow accepted geotechnical analysis practice. In the case of piles driven to partially weathered rock, however, the toe model in MultiPier did not appear to be stiff enough. Even with values of initial or low strain shear modulus that have been measured in typical residual soils, significant displacement is required before the ultimate toe capacity is reached. As such, a very stiff toe model was adopted to model end bearing in weathered rock or rock. Further research should be pursued to either determine model parameters that work in the built-in approach or develop an entirely new model that better captures the behavior of soils in which piles in North Carolina typically bear.

Recommendations for soil stiffness parameters for use in the MultiPier were also made. The low strain shear moduli presented in Figures 8 and 9 by Borden and Shao (1995) and Hardin and Black (1968) yielded much more reasonable side and toe resistance versus displacement curves than the high strain shear modulus values suggested in the MultiPier manual (BSI, 2000).

The results from MultiPier and LPILE were similar when a single pile nonlinear analysis was performed. Similarly, from a structural engineering standpoint, the verification analyses conducted with SAP indicated that MultiPier can capture nonlinearity of reinforced concrete elements. Slight differences in the results were observed due to a lack of sufficient resolution in the displayed MultiPier P-y and t-z curves in the early, highly nonlinear portion of the curves. As these curves were used as input data into SAP, in effect, the resolution issue led to lower stiffness, and thus higher displacements from SAP compared to MultiPier. The differences, however, tended to be relatively small.

Nonlinear 3-D Analysis Results

The four bridges modeled in this study included single row H-Pile, pipe pile and prestressed concrete pile bents, as well as one two row prestressed concrete pile bent. These were modeled with 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. Overall, all four bridge models showed good agreement between MultiPier and SAP, which served to verify the MultiPier results and provide a better understanding of the behavior of free-standing pile bents in general..

Once the nonlinear models were set, MultiPier was used to investigate the possibility of optimizing the pile bent design by reducing the number of pile elements. In these cases, the “pile optimization” analyses were performed solely considering the yield strengths of the pile and bent cap materials, as well as the displacements induced by the applied factored loads. Construction aspects (such as pile driveability with existing contractor equipment), redundancy requirements, and aesthetic decisions were purposefully not considered since they are outside the purview of this study. After the analyses were complete, the NCDOT Bid Avereages from 2004 were used to estimate a cost savings per bent by comparing the as-built (Georgia Pier designed) cost with the cost of building the more structurally efficient bent. The bid averages database is used for estimating purposes by the NCDOT when jobs are in the bid stage.

For the Robeson county H-Pile bridge bent, it was shown that 8 HP14x73 piles could be reduced to as few as 5 HP12x53piles, for an estimated cost savings of up to $10,400 per bent. For the Northampton county pipe pile bridge bent, five 24 inch diameter closed end pipe piles could be reduced to four piles for an estimated savings of $8,700 per bent or reduced in size to five 18 inch diameter pipes for an estimated savings of $15,600 per bent. The eight 18 inch square prestressed concrete pile bent from Halifax County could be reduced to as few as six piles, at an estimated cost savings of $5,400 per bent. A similar cost saving was observed when the concrete piles were replaced by eight HP14x73 piles. Finally, the Washington County Bridge, built with two rows of five 16 inch square concrete piles, could be reduced to two rows of four piles, at an estimated savings of $5,100 per bent.

In the above economic calculations, the savings gets incrementally larger as the number of bents increases. For the Robeson, Northampton and Washington County Bridges, which each have only one or two bents, these savings are significant but not enormous. For the Halifax County’s bridge, however, which has nine interior bents, the cost savings gets more significant. Thus, the nonlinear analysis shows its value as the number of bridge bents, and thus the complexity of the structure increases.

From the analyses results of the case studies, the following was observed:

1. Lateral deflection in the longitudinal direction is most sensitive to differences in soil modeling approaches and will tend to control the design if a free standing pile bent is assumed and displacement limit states are a consideration in the design. For a free head assumption, only the soil is preventing piles from displacing. In the transverse direction, the relatively rigid cap beam also prevents large scale displacements. Determining whether intermediate types of fixity (e.g. "partially fixed," "partially free" or some other restraint on displacement or rotation at the pile top) are applicable will require more study of full bridge models.
2. The free standing pile bent is a "worst case" scenario from a displacement standpoint. If a connection to the superstructure is not modeled, any load transferred through the connections and superstructure to the abutments is considered to act on the pile bent only. Thus, a maximum load is applied to the bent cap, and therefore maximum displacements will be obtained.
3. Although it was not an issue for the four bridges investigated, axial deformation of the piles plays 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.
4. If nonlinear analysis is used as design tool, it is best to model the piles as nonlinear sections, instead of linear elastic sections. In these cases, the section dimensions of steel piles or the distribution of reinforcement in, and geometry of, prestressed concrete piles are standardized As nonlinear sections, programs like MultiPier can calculate demand/capacity ratios and perform more sophisticated numerical analyses, particularly for concrete piles, all of which should lead to better design data. In addition, nonlinear soil models should be used. The cap beam, however, can be modeled as a linear section with a moment of inertia for cracked concrete. Once initial moments and shears are calculated in the section, rebar can be sized. If needed, a new cracked moment of inertia could be recalculated with the new rebar. This requires pre and post processors to help automate load generation and rebar design.
5. The potential for cost savings was analyzed using NCDOT bid averages. The results showed the possibility of cost savings as the number of piles were reduced while maintaining demand/capacity ratios of less than 1.0 and displacements of

approximately one inch or less. It should be mentioned, however, that the cost savings analysis did not take into account issues such as contractor's pile installation equipment limitations or axial pile capacity limits currently in use by NCDOT.

Equivalent Models

Two equivalent models were developed as linear frame analyses. These models had piles represented by an equivalent length, using a point of fixity approach. Both models were implemented in SAP2000—one was based on the same assumptions as Georgia Pier, (DOT POF) while the other was developed to provide equivalent length based on results from a nonlinear single pile lateral analysis.

The proposed method of calculating a point of fixity uses the moments and displacements calculated from a single pile lateral analysis, such as those available in LPILE or MultiPier. For accurate results, this lateral analysis must use the highest applied lateral load and its corresponding axial load to determine the maximum moment and horizontal displacement experienced by the pile. These values are then used to calculate a length fixity based on matching moments, and matching displacements through inertia reduction factors. To capture the disparate behavior in the transverse and lateral directions, single pile analyses can be run twice: once with a free head condition (longitudinal) and once with a fixed head condition (transverse). This approach, however, leads to multiple points of fixity. If displacements are however not an issue, point of fixity based on inertia reduction factors can be ignored.

The proposed equivalent point of fixity model was created to closely match the results of the full nonlinear single pile lateral analysis for a given loading case. To compare elastic cantilever models, the DOT POF and proposed equivalent (i.e. two point of fixity with reduction factor) models were applied to a single cantilevered pile (in an attempt to more simply approximate an LPILE analysis) in each of the four case studies. The results presented in Table 25 showed that in the transverse direction, the proposed equivalent formulation predicts on average 50% of the displacement, 70% of the maximum moment, and 220% more axial-buckling capacity than the DOT POF model. In the longitudinal direction, the proposed equivalent formulation on average predicts 50% of the displacement, 55% of the maximum moment, and 375% more axial-buckling capacity than the DOT POF model. Of course, these averages are limited to the four load cases, soil conditions and pile types studied for this analysis.

The results of the nonlinear, proposed equivalent, and DOT POF (a recreation of Georgia Pier in SAP) models, were compared for the four bridges studied with results presented in Tables 26 to 29. In all cases, when displacements are modeled and compared to the equivalent and nonlinear models, the DOT POF method predicts lateral displacements in both the transverse and longitudinal directions that are twice as large as the deeper point of fixity used in Georgia Pier. Because the joints are fixed against vertical translation, axial displacements at the pile tops are lower (at zero inches) and moments in the cap beam tend to be slightly lower in Georgia Pier than in the other models, as well. Axial forces are slightly higher in the Georgia Pier model. The following additional observations are made:

1. The NCDOT POF model definition is conservative. In this study, the deeper point of fixity in all four bridges lead to longer cantilever pile lengths. This in turn leads to higher predicted displacements, slightly higher forces in the pile, and higher moments in the cap beam. The higher moments were not necessarily observed from the analysis, however, since Georgia Pier artificially fixed the vertical displacement at the pile top. This limitation led to reducing the moments since no differential vertical displacement is allowed.
2. The proposed equivalent model gives results that are closer to the results of the nonlinear analysis for the maximum lateral loads applied to the structure. For smaller loads, the results from the equivalent frame will be conservative, although less so than from the current NCDOT's approach. The proposed approach requires a nonlinear single pile analysis (such as those obtained using LPILE or MultiPier) performed with a similar lateral load and pile top fixity as the piles in the bridge in question. The equivalent model is then valid for a specific load level. Given the results of the output, equivalent length, k factors and stiffness reduction factors can be determined, as described in Chapter 5. Once these values are placed in an equivalent frame analysis model, the moment, force, and displacement in the piles and bent cap should be similar to those obtained from the nonlinear model.
3. One drawback of the proposed equivalent frame model is that it requires a few more calculated quantities than the current NCDOT practice. It also requires a frame program that can mimic a two point of fixity system by restraining motion in the direction in which the shorter point of fixity is needed. If MultiPier is used for the single pile lateral analysis, it will likely not be considerably more effort to model the entire bridge bent and there will be no need for the equivalent point of fixity model.

4. The equivalent model may be best suited for small or simpler structures, where additional considerations of a nonlinear analysis are not warranted.
5. The equivalent model may also be useful as a simple check on the nonlinear analyses. If a result does not appear to make sense, the large number of input parameters in a nonlinear model will make it difficult to find the error. An equivalent model could provide a convenient check on a more complicated structural analysis program.

Displacement Limit States

Generally, there is a lack of clearly defined limit states for bridge bents in the literature. Moulton's 1986 study identified several long term displacements that caused intolerable damage in a number of highway bridges. Similarly, Liu ***et al.***'s 2001 NCHRP study suggested a simple displacement limit of the 2% of the free column height, which corresponded to a point defining the onset of plastic behavior. However, evaluation of the limit states for substructures includes aspect ratio, type of pile material, level of damage that are typically defined by strains, and impact of the soil yield level.

A brief survey of possible geotechnical limit states was conducted. The gap formed by the maximum lateral displacement of the pile at the ground surface was considered, as well as loss of axial shaft resistance that may occur due to opening of the gap. Based on these analyses, it was concluded that only piles that develop most of their capacity from shaft resistance would be affected from loss of contact in between the sides of the pile and the soil. Predominantly end bearing piles, like those designed by the NCDOT in weathered rock or rock bearing strata, were largely unaffected.

A separate set of pushover analyses were conducted on the SAP pile bent models with nonlinear piles and soil elements to consider the NCDOT's "one inch" lateral displacement limit imposed during the lateral pile analyses. The study sought to determine at what pile bent displacement elastic yielding in the pile occurred. Keeping in mind that the scope of this study is limited by the small number (4) of bridges analyzed, it would appear that, for transversely loaded bridge bents, lateral deflection of the piles can be increased to two inches before yielding occurs. For a free standing bent loaded longitudinally, two inches is a conservative value for the four bridges studied. Similarly, the simple lateral displacement limit of 2% of free column height suggested by Liu et al. (2001) was conservative for the four bridges studied in both the transverse and lateral direction. For more general application, however, further cases should be examined that consider a typical range of soil types, pile and free column height conditions in North Carolina.

Substructure-Superstructure interaction was considered using a kinematic model of one bridge. Based on this preliminary work, it appears a significant amount of load is transferred away from the pile bent and into the abutments. The stiffness of the superstructure makes it behave like a rigid body as it rotates and translates. Accordingly, the limit state for closing expansion joints can be evaluated using simple geometric analyses. A series of mathematical models for different bridge configurations was developed. A numerical example showed the evaluation of lateral forces and displacements required for expansion joint closure. In the example case, the four inch displacement calculated likely would have brought about failure in the pile bent, at least for prestressed concrete piles.

Recommendations

Based on analyses and results obtained in this study, the following recommendations are made:

1. Update software program used in design. Georgia Pier has its limitations and a change to a more comprehensive frame analysis program or to a nonlinear 3-D analysis program is recommended. If a frame analysis model is selected, the equivalent length method described herein is recommended.
2. If a nonlinear analysis program such as MultiPier is used, pre and post processors are needed in order to fully automate the design process.
3. Proposed Possible Design Procedures are as follows:

I. Equivalent Frame analysis

i. Determine soil parameters,

ii. Identify preliminary pile design (pile type, length, allowable capacity),

iii. Perform single pile lateral analysis using assumed or preliminary loads (Geotechnical Unit),

iv. Determine equivalent model parameters (Structures or Geotechnical Unit),

v. Create model in frame analysis software (Structures Unit) and run analysis

vi. Compare displacements, moments and loads to initial single pile analysis results. Rerun the single pile analysis with revised loads if the preliminary loads are greater than **20%** of the analysis results or if the preliminary loads are lower than the analysis results. This may require several iterations and communication between the two Units.

vii. Consider displacements, and size sections for moments and loads

viii. Design cap beam reinforcement

ix. Check capacity of piles

:

II. Nonlinear 3-D analyses

i. Identify preliminary design (pile type, superstructure)

ii. Estimate initial loading conditions (preprocessor)

iii. Determine soil model parameters and initial pile type and length (Geotechnical Unit)

iv. Enter pile and soil (Geotechnical Unit), then, linear bent cap parameters, bearing locations and load conditions into model (Structures Unit)

v. Run analysis

vi. Consider displacements, size sections for moments and loads

vii. Design cap beam reinforcement

viii. Check capacity of piles

ix. Double check loading assumptions used in initial analysis (maximum lateral and axial load) by comparing to loads experienced by the piles in the frame analysis. If the initial loads are incorrect based on changes to the superstructure or substructure geometry in the design, recalculate initial loadings and run process again. In this case, the design load checks could be kept within the Structures Unit, unless significant changes to the pile size or type required the Geotechnical Unit to check again.

As noted earlier, it may be most prudent to use nonlinear analyses where the greatest cost savings are possible—i.e. in bridges with a significant number of bents or where complex foundation conditions exist. As far as input data requirements are concerned, the two methods are quite similar: the soil parameters required for the lateral pile analysis in the equivalent models are basically the same as those required for the nonlinear models. Using the equivalent models may take a slightly more effort since additional "equivalent" values must be determined. The current challenge however with the nonlinear programs is their lack of modules to assist with design and load entry. This current limitation is a hurdle for day to day use in a design office.]

Future Research

The following areas are identified as requiring additional work:

1. An extension of this project to determine the applicability of these models to drilled shaft piers and monopole bents would naturally follow from this project. A variety of drilled shaft foundations, bent cap types and superstructures could be modeled in a manner similar to that presented here.

2. As noted previously, more work is needed on selecting the proper toe model for the piles, particularly if axial displacement is considered. This could be performed by studying instrumented static load tests in various regions of the state and either directly using the toe response or trying to determine the toe response based on back calculation from static or dynamic testing results.

3. The connections of the superstructure to the substructure should be considered to more accurately model the entire bridge. This includes evaluating the elastomeric bearing pads, as well as studying the response of typical types of abutments. These model parameters should also help improve the displacement limit state analyses begun in this report.

4. This study considered standalone single pile bents. Future work should look at the impact of modeling the entire bridge as an abutment-superstructure-substructure-soil interaction problem.

5. The selected design model should be prepared for future use of LRFD, which is already the case for MultiPier. This would include sizing of all structural members, as well as geotechnical design of the foundations. The preprocessors for load determination would need to be updated as well.

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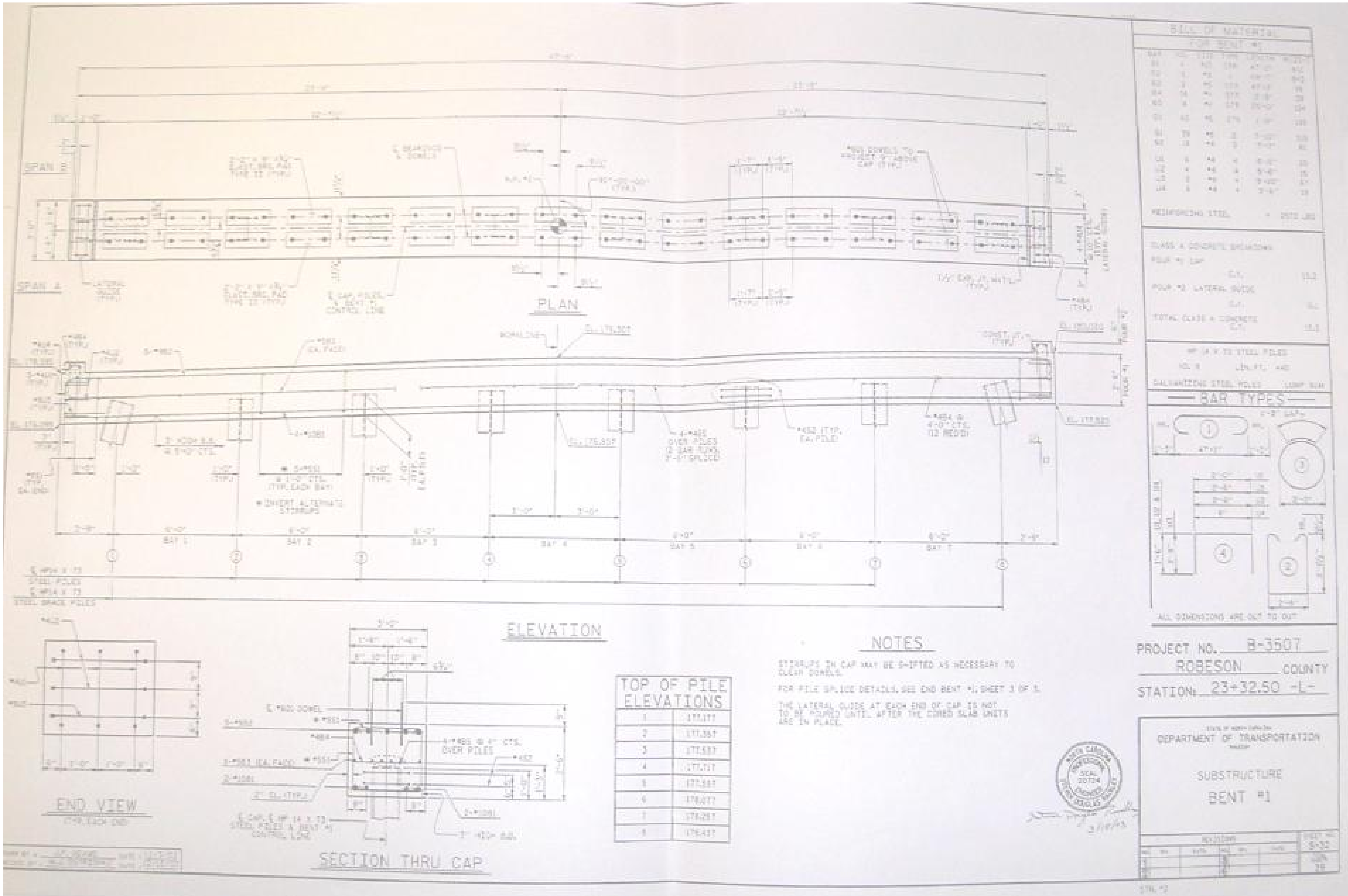
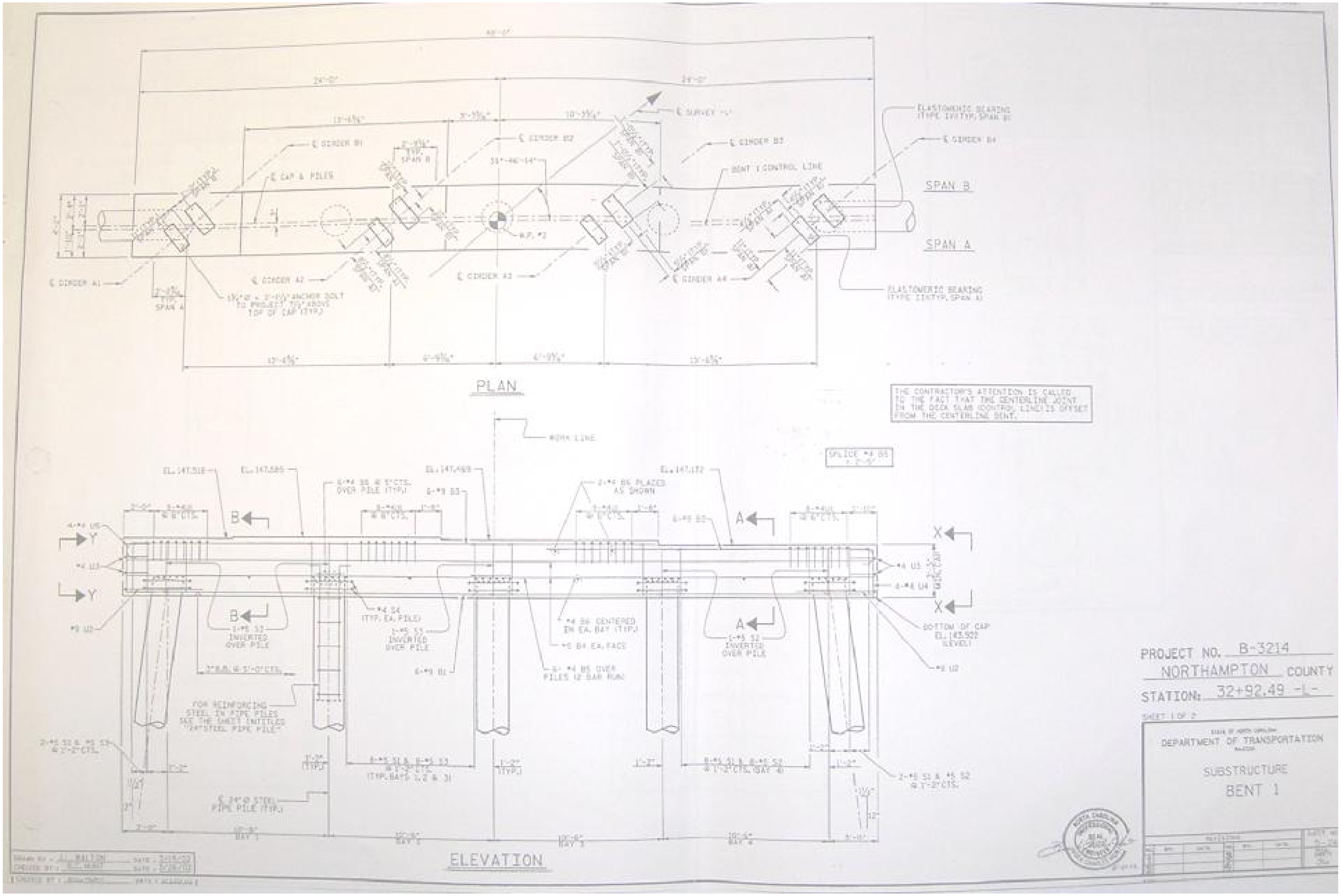
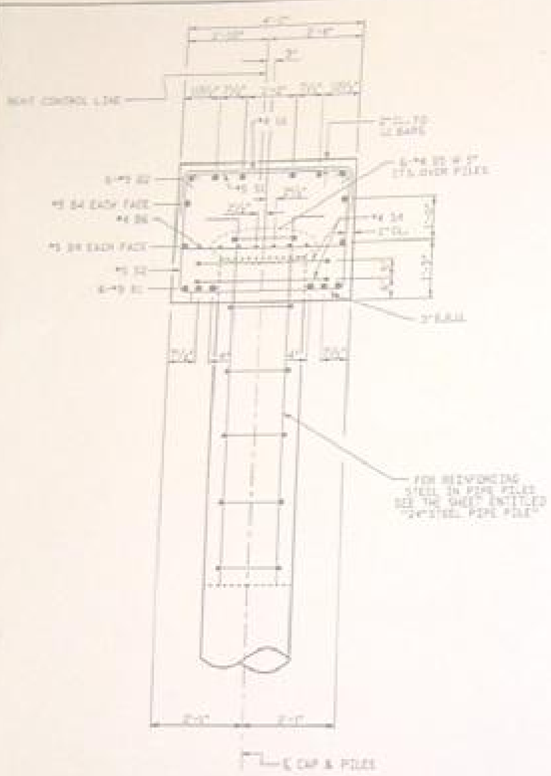
Appendix A

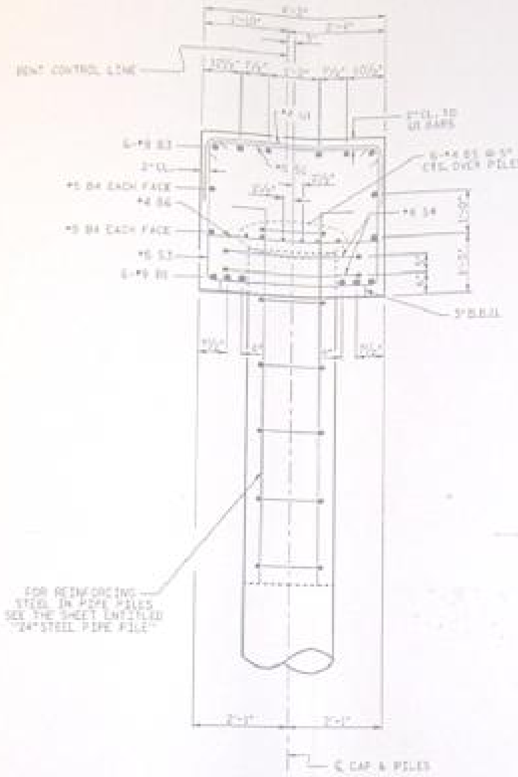
Selected Drawings from the NCDOT for Four Bridge Case Studies


BILL OF MATERIAL
FOR BENT #1
SPAN B
SPAN A
PLAN
ELEVATION
BAY 1
BAY 2
BAY 3
BAY 4
BAY 5
BAY 6
BAY 7
BAR TYPES
ALL DIMENSIONS ARE OUT TO OUT
END VIEW
SECTION THRU CAP
NOTES
STIRRUPS IN CAP MAY BE SHIFTED AS NECESSARY TO CLEAR DOWELS.
FOR PILE SPLICE DETAILS, SEE END BENT #1, SHEET 3 OF 3.
THE LATERAL GUIDE AT EACH END OF CAP IS NOT TO BE POURED UNTIL AFTER THE CORED SLAB UNITS ARE IN PLACE.
TOP OF PILE ELEVATIONS
1 177.177
2 177.357
3 177.537
4 177.717
5 177.897
6 178.077
7 178.257
8 178.437
PROJECT NO. B-3507
ROBESON COUNTY
STATION: 23+32.50 -L-
DEPARTMENT OF TRANSPORTATION
SUBSTRUCTURE
BENT #1
REVISIONS
S-32
STR. #2

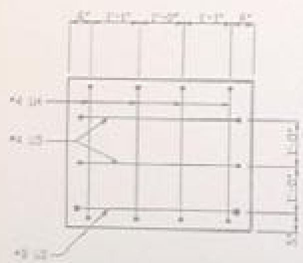

PLAN
ELEVATION
SPAN B
SPAN A
WORK LINE
THE CONTRACTOR'S ATTENTION IS CALLED TO THE FACT THAT THE CENTERLINE JOINT IN THE DECK SLAB (CONTROL LINE) IS OFFSET FROM THE CENTERLINE BENT.
PROJECT NO. B-3214
NORTHAMPTON COUNTY
STATION: 32+92.49 -L-
SHEET 1 OF 2
DEPARTMENT OF TRANSPORTATION
SUBSTRUCTURE
BENT 1



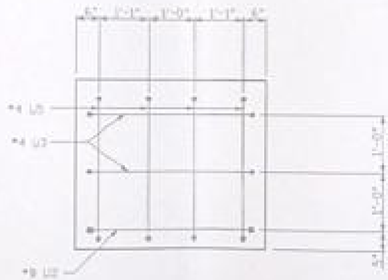
SECTION A-A



SECTION B-B

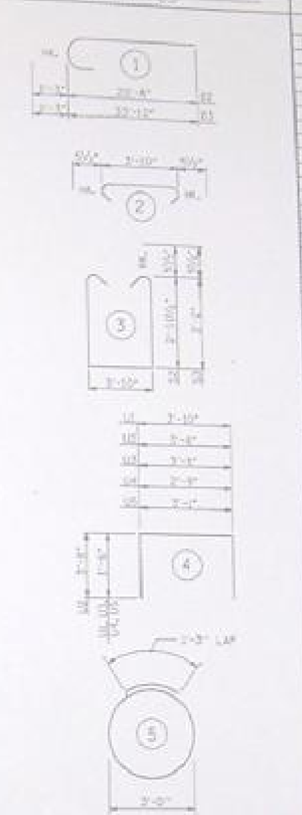


VIEW X-X



VIEW Y-Y

BAR TYPES



ALL BAR DIMENSIONS ARE OUT TO OUT

BILL OF MATERIAL

BENT 1

BAR	NO.	SIZE	TYPE	LENGTH	WEIGHT
B1	6	#9	STR	47'-8"	972
B2	6	#9	1	25'-11"	447
B3	6	#9	1	26'-1"	[illegible]
B4	4	#5	STR	47'-8"	199
B5	6	#4	STR	26'-1"	210
B6	18	#4	STR	3'-10"	47
S1	36	#5	2	4'-9"	178
S2	12	#5	3	10'-6"	131
S3	28	#5	3	10'-3"	335
S4	10	#4	3	10'-9"	72
U1	34	#4	4	6'-10"	146
U2	2	#9	4	11'-0"	75
U3	4	#4	4	6'-9"	18
U4	4	#4	4	5'-9"	15
U5	4	#4	4	6'-1"	16
REINFORCING STEEL				LBS.	3658

CLASS 'A' CONCRETE

* CAP (POUR #1) C.Y. 25.4

TOTAL C.Y. 25.4

PP 24 x 0.5 STEEL PILES

NO. 5 LIN.FT. 300.0

* NOTE: CONCRETE DISPLACED BY THE 24" STEEL PIPE PILES HAS BEEN DEDUCTED.

PROJECT NO. B-3214

NORTHAMPTON COUNTY

STATION: 32+92.49 -L-

SHEET 2 OF 2

DEPARTMENT OF TRANSPORTATION

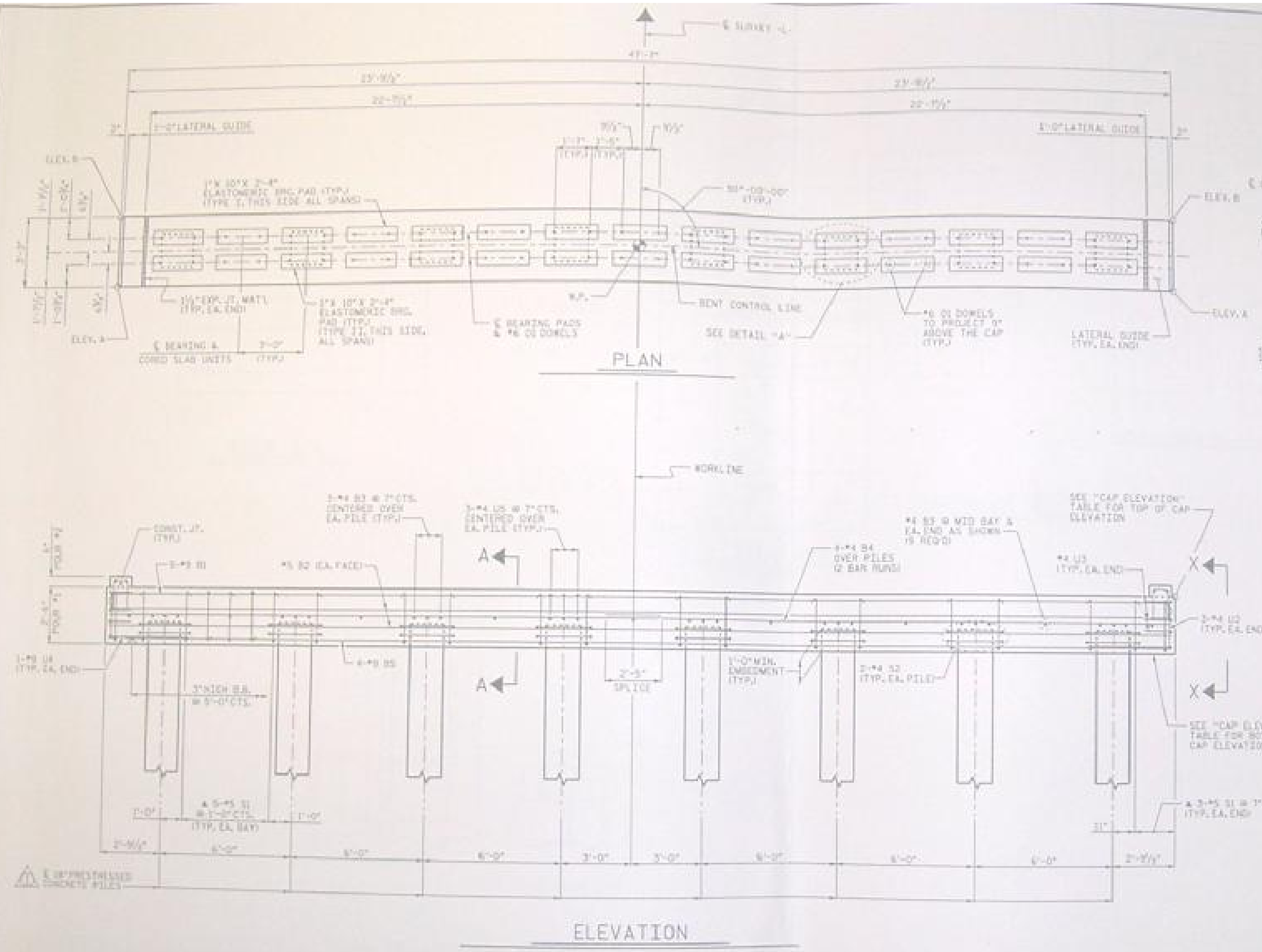
SUBSTRUCTURE

BENT 1

REVISIONS

NO.	BY	DATE	NO.	BY	DATE
1			3		
2			4		

SHEET NO. S-11



NOTES

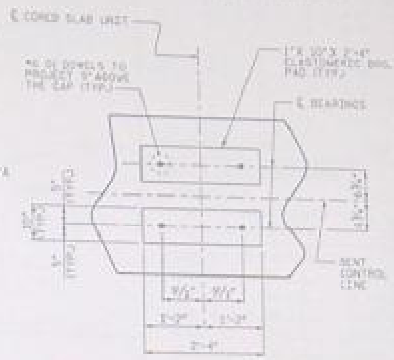
STIRRUPS AND "U" BARS IN CAP MAY BE SHIFTED AS NECESSARY TO CLEAR DOWELS.

CONCRETE DISPLACED BY THE 18" SQUARE PRESTRESSED CONCRETE PILES HAS BEEN DEDUCTED FROM THE CONCRETE QUANTITY.

▲ INVERT ALTERNATE STIRRUPS AS SHOWN.

THE LATERAL GUIDES ARE NOT TO BE POURED UNTIL AFTER THE CORED SLAB UNITS ARE IN PLACE.

FOR LATERAL GUIDE DETAIL, SEE SHEET 2 OF 2.



DETAIL "A"

(DIMENSIONS ARE TYPICAL EACH BENT)

CAP ELEVATIONS			
BENT NO.	TOP OF CAP		BOTTOM OF CAP ELEVATION
	ELEV. A	ELEV. B	
2	86.277	86.277	83.777
3	86.310	86.310	83.810
4	86.234	86.234	83.734
5	86.219	86.200	83.700
6	86.065	86.065	83.565
7	85.915	85.915	83.415
8	85.653	85.627	83.127

PROJECT NO. B-2980

HALIFAX COUNTY

STATION: 22+34.00 -L-

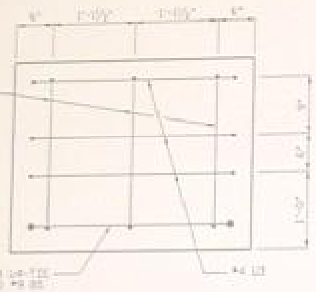
SHEET 1 OF 2

DEPARTMENT OF TRANSPORTATION

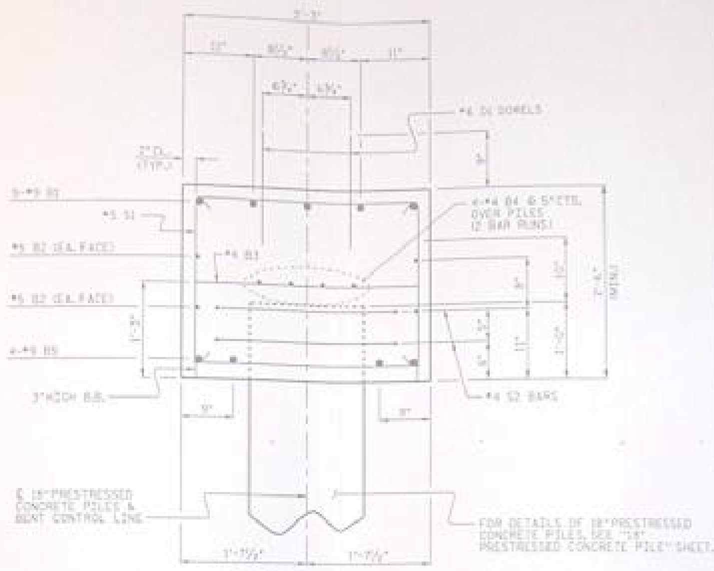
SUBSTRUCTURE

BENT NO. 2-8

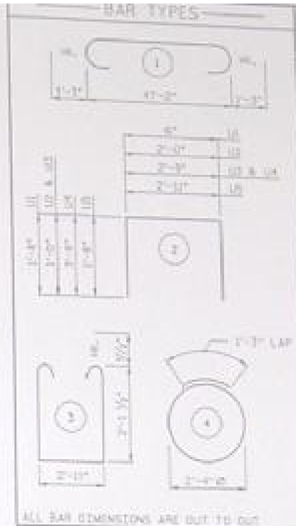
REVISION No. 1
CHANGED PP 16 X 0.50 STEEL PILES TO 18" PRESTRESSED CONCRETE PILES (PER CONTRACTOR'S REQUEST)
BY: PCB CHECKED BY: JWR



VIEW X-X
(TYP. EA. END AND EA. BENT)



SECTION A-A
(TYP. EA. BENT)



BILL OF MATERIAL
FOR ONE BENT
(7 BENTS REQUIRED)

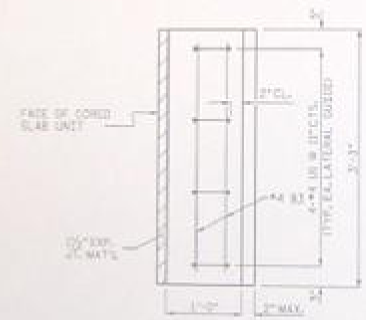
BAR	NO.	SIZE	TYPE	LENGTH	WEIGHT
B1	6	9	1	49'-8"	844
B2	4	5	STR	47'-3"	197
B3	37	4	STR	2'-11"	72
B4	8	6	STR	24'-10"	159
B5	4	9	STR	47'-3"	643
D1	60	6	STR	1'-6"	135
S1	41	5	3	8'-1"	346
S2	16	4	4	6'-7"	70
U1	8	4	2	3'-3"	17
U2	6	4	2	4'-0"	16
U3	6	4	2	4'-9"	19
U4	2	9	2	10'-1"	69
U5	24	4	2	6'-3"	100
REINFORCING STEEL					2683 LBS.

CLASS A CONCRETE BREAKDOWN

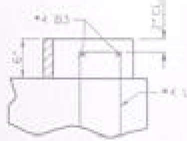
POUR #1 (CAP)	13.7 C.Y.
POUR #2 (LATERAL GUIDES)	0.1 C.Y.
TOTAL CLASS A CONCRETE	13.8 C.Y.

18" PRESTRESSED CONCRETE PILES

BENT NO. 2	NO. 8	LIN. FT. 360
BENT NO. 3	NO. 8	LIN. FT. 360
BENT NO. 4	NO. 8	LIN. FT. 400
BENT NO. 5	NO. 8	LIN. FT. 400
BENT NO. 6	NO. 8	LIN. FT. 400
BENT NO. 7	NO. 8	LIN. FT. 400
BENT NO. 8	NO. 8	LIN. FT. 360



PLAN



ELEVATION

PLAN OF LATERAL GUIDE
(RIGHT LATERAL GUIDE SHOWN, LEFT LATERAL GUIDE SIMILAR)
(TYP. EA. BENT)

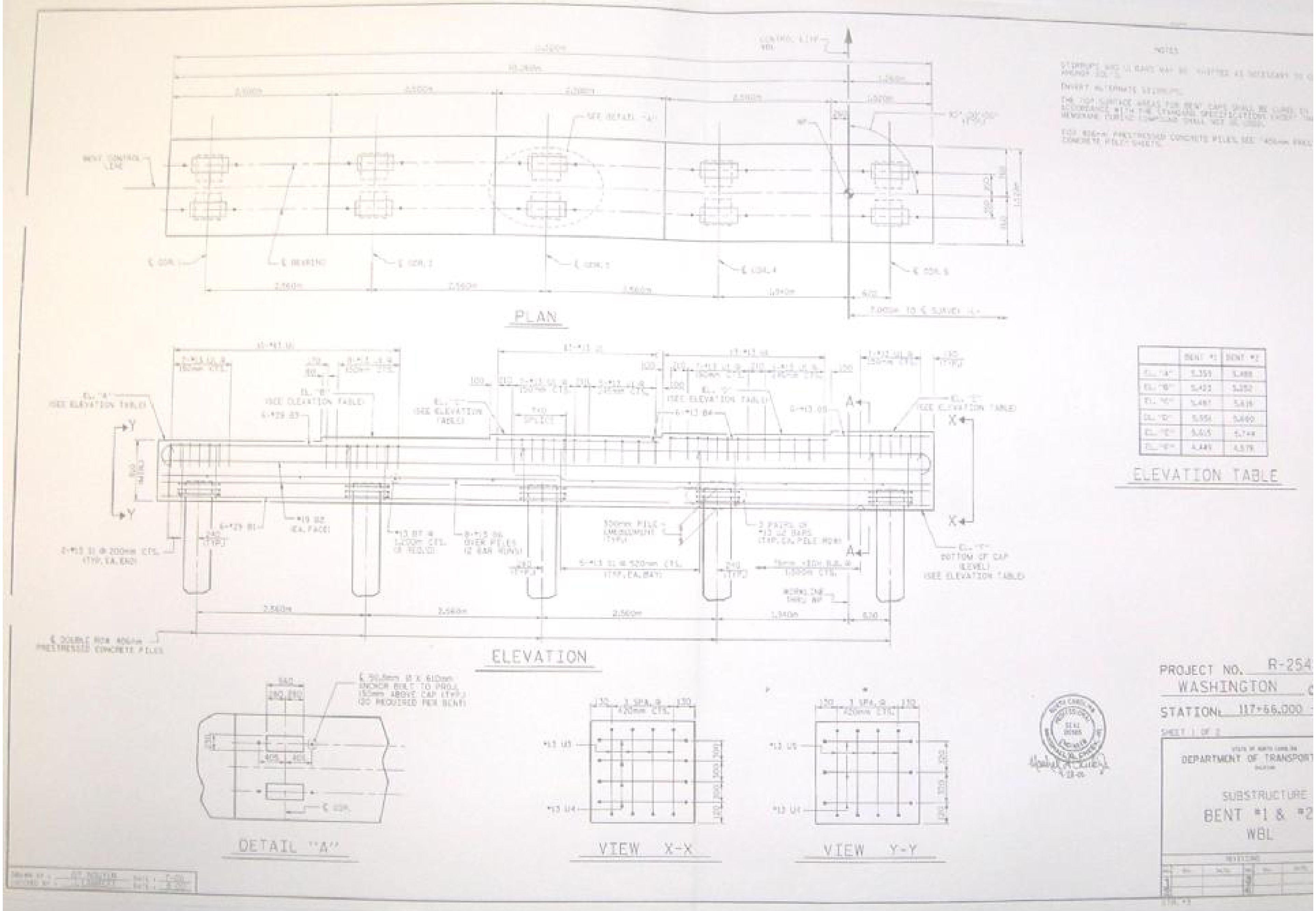
REVISION No. 1
CHANGED PP 16 X 0.50 STEEL PILES TO 18" PRESTRESSED CONCRETE PILES.
(PER CONTRACTOR'S REQUEST)
BY: PCB CHECKED BY: JWR

PROJECT NO. B-2980
HALIFAX COUNTY
STATION: 22+34.00 -L-

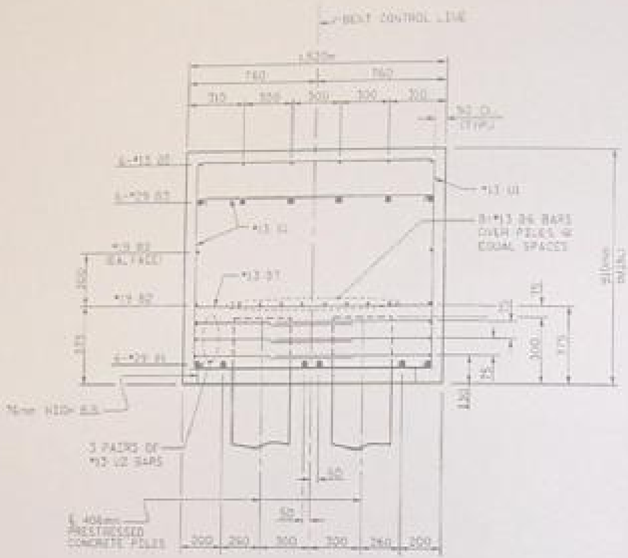
SHEET 2 OF 2

DEPARTMENT OF TRANSPORTATION

SUBSTRUCTURE
BENT NO. 2-8

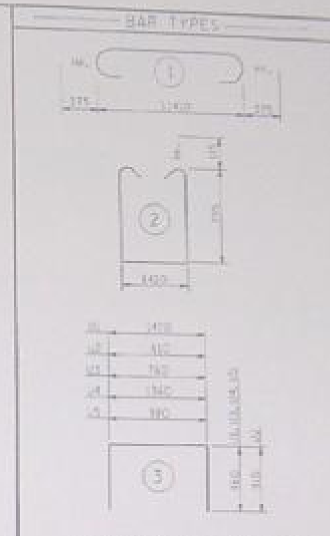


NOTES
PLAN
ELEVATION
ELEVATION TABLE
DETAIL "A"
VIEW X-X
VIEW Y-Y
PROJECT NO. R-254
WASHINGTON
STATION: 117+66.000
DEPARTMENT OF TRANSPORT
SUBSTRUCTURE
BENT #1 & #2
WBL



SECTION A-A

BAR TYPES



ALL BAR DIMENSIONS ARE OUT TO OUT.

BILL OF MATERIAL					
BENT #1 & #2					
ONE BENT SHOWN (2 REQUIRED)					
BAR	NO.	SIZE	TYPE	LENGTH	WEIGHT
B1	6	#29	STR	3350	147
B2	4	#13	STR	1420	[illegible]
B3	6	#29	1	12410	384
B4	12	#13	STR	1410	19
B5	6	#13	STR	1420	8
B6	16	#13	STR	6050	97
B7	6	#13	STR	1420	8
S1	24	#13	2	3800	91
U1	48	#13	3	2340	112
U2	30	#13	3	2440	73
U3	6	#13	3	1650	9
U4	7	#13	3	2290	16
U5	6	#13	3	1420	8
REINFORCING STEEL			KG		1156
CLASS A CONCRETE CU. METERS					17.3
406mm PRESTRESSED CONCRETE PILES					
BENT #1	NO. 10		METERS		130
BENT #2	NO. 10		METERS		140

* NOTES: CONCRETE DISPLACED BY 406mm PRESTRESSED CONCRETE PILES HAS BEEN DEDUCTED.

PROJECT NO. R-2548B

WASHINGTON COUNTY

STATION: 117+66.000 -L-

SHEET 2 OF 2

STATE OF NORTH CAROLINA
DEPARTMENT OF TRANSPORTATION
RALEIGH

SUBSTRUCTURE
BENT #1 & #2
WBL