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Development of abutment design standards for local bridge designs

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Development of abutment design standards for local bridge designs

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ABSTRACT

Several superstructure design methodologies have been developed for low volume road bridges by the Iowa State University Bridge Engineering Center. However, to date no standard abutment designs have been developed. Thus, there was a need to establish an easy to use design methodology in addition to generating generic abutment standards and other design aids for the more common systems used by Iowa counties.

The summary of this investigation is divided into two sections. The first section, which consists of Chapters 1 through 9 and Appendices A and B, summarizes the research completed for this project. A similar version of this first section is also published as Volume I of the Iowa Department of Transportation Project TR-486 final report. A survey of the Iowa County Engineers was conducted and it was discovered that while most counties use a similar type of abutment, only 17 percent use some type of standard abutment design or plans. A literature review revealed several possible alternative abutment systems for future use on low volume road bridges in addition to two separate substructure lateral load analysis techniques. The lateral load techniques consisted of a linear and non-linear method. The linear analysis method was used for this project based on the relative simplicity and the accuracy of the maximum pile moment when compared to the non-linear analysis method. The resulting design methodology was developed for single span stub abutments supported on steel or timber piles with a span length range of 20 to 90 ft and roadway widths of 24 and 30 ft. However, other roadway widths can be designed using the foundation design template provided. The backwall height is limited a range of 6 and 12 ft and the soil must be described as a cohesive or cohesionless soil. The design methodology was developed using the guidelines specified by the American Association of State Highway Transportation Officials Standard Specifications, the Iowa Department of Transportation Bridge Design Manual, the American Institute of Steel Construction Manual of Steel Construction, and the National Design Specifications for Wood Construction.

The second section, which is presented as Appendix C, introduces and outlines the use of the design aids developed for this project. It should be noted that a similar version of Appendix C is also published as Volume II of the TR-486 final report. Charts for determining dead and live gravity loads based on the roadway width, span length, and superstructure type are provided. A foundation design template was developed in which the engineer can check a substructure design by inputting basic bridge site information. Tables published by the Iowa Department of Transportation that provide values for estimating pile friction and end bearing for different combinations of soils and pile types were also summarized. Generic standard abutment plans were developed in which the engineer can complete necessary bridge site information in the spaces provided. These tools enable engineers to design county bridge substructures more efficiently.

1. INTRODUCTION

1.1. BACKGROUND

In the 1994 Iowa Highway Research Board (Iowa HRB) Project HR-365 several replacement bridges being used by the counties of Iowa in addition to the surrounding states were identified, reviewed and evaluated [1]. Results for a survey of the Iowa County Engineers and neighboring states indicates that:

- Sixty-nine percent of the Iowa counties have the capabilities to construct relatively short spans bridges with their own forces.
- The most commonly used replacement bridges are continuous concrete slabs and prestressed concrete girder bridges for the primary reason that standard designs are readily available and have minimal maintenance requirements.
- There are several unique replacement bridge systems that are constructed by county forces.
- Two bridges systems were identified for additional investigation.

The development of the first system, Steel Beam Precast Units, started in the Iowa HRB Project HR-382 [2, 3]. The Steel Beam Precast Unit concept involves the fabrication of a precast unit constructed by county forces. The precast units are composed of two steel beams connected by a composite concrete slab. The deck thickness is limited to reduce unit weight so that the units can be fabricated off site and then transported to the bridge site. Once at the bridge site, adjacent precast units are connected and the remaining portion of the concrete deck is placed. A Steel Beam Precast Unit demonstration bridge was constructed and tested along with the development of design software, a set of completed designs for a range of roadway widths and span lengths, and generic plans.

The development of the second bridge system which involves the modification of the Benton County Beam-in-Slab Bridge (BISB), TR-467 [4], is currently in progress. The cross-section of the original BISB system and the modified BISB system are shown in Figures 1.1 and 1.2, respectively. The basic differences in the two systems are the removal of the structurally ineffective concrete from the tension side of the cross-section and the addition of an alternate shear connector. The alternate shear connector was developed as a part of HR-382 to create composite action between the steel beams and the concrete. These two modifications decrease the superstructure dead load and improve the structural efficiency thus allowing the modified BISB to span greater lengths. Upon the completion of TR-467, a design methodology will be developed along with a generic set of plans for the system.

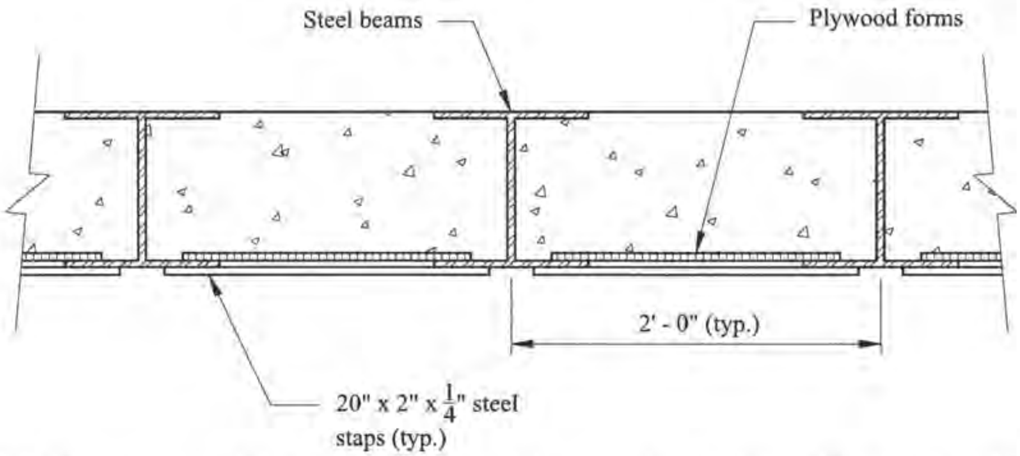


Figure 1.1. Cross-section of the original beam-in-slab system [adapted from Klaiber et. al, 2004].

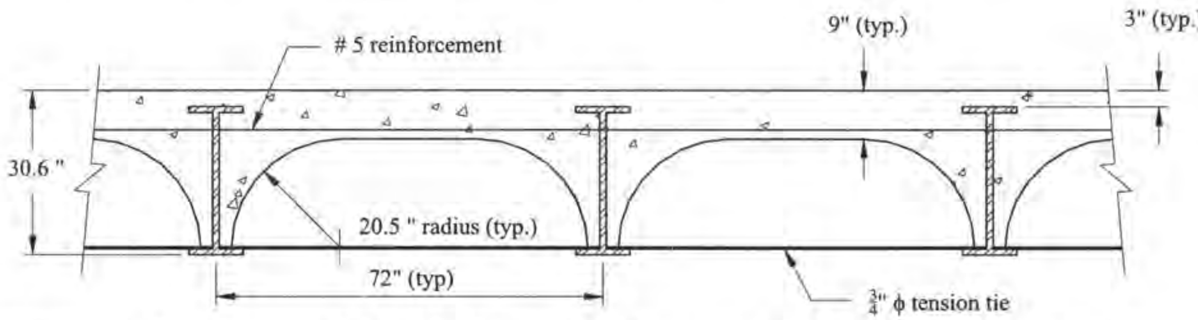


Figure 1.2. Cross section of a modified beam-in-slab system [adapted from Klaiber et. al, 2004].

In Iowa HRB Project TR-444 [5], a rail road flatcar (RRFC) superstructure system for low-volume Iowa county roads was developed. This project involved inspecting various decommissioned RRFC's for use in demonstration bridges, the construction and laboratory testing of a longitudinal joint between adjacent RRFC's, the design and construction of two RRFC demonstration bridges, and development of design recommendations for future RRFC bridges. The cross-section of a three-span RRFC bridge (total length = 89 ft) built in Winnebago County, Iowa in 2002 is presented in Figure 1.3, while the cross-section of the single-span RRFC bridge (total length = 56 ft) built in Buchanan County, Iowa in 2002 is presented in Figure 1.4.

As previously noted, various superstructure design methodologies have been developed by the Iowa State University (ISU) Bridge Engineering Center (BEC), however to date no standard abutment designs have been developed. Obviously with a set of abutment standards and the various superstructures previously developed, a county engineer could design the complete bridge at a given location. Thus there is a need to establish an easy to use design methodology in addition to generating generic abutment standards for the more common systems used in Iowa counties.

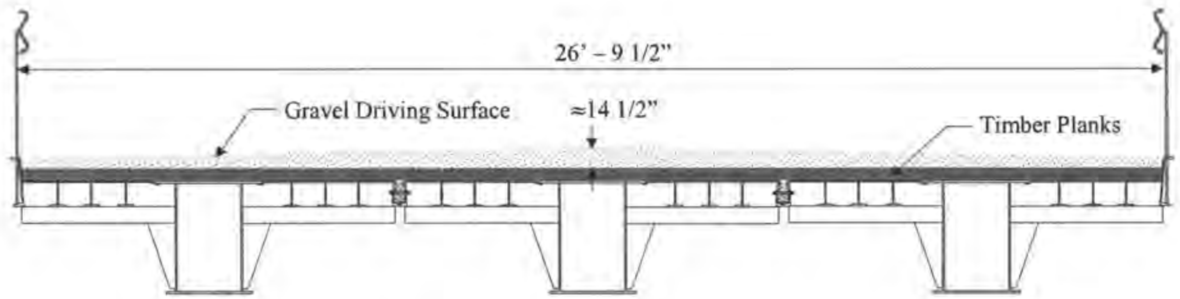


Figure 1.3. Cross-section of the Winnebago County Bridge [adapted from Wipf et al., 2003].

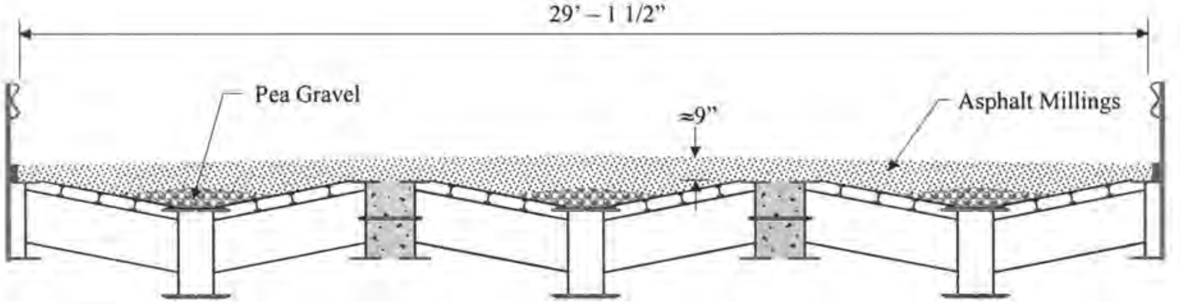


Figure 1.4. Cross-section of the Buchanan County Bridge [adapted from Wipf et al., 2003].

1.2. OBJECTIVE AND SCOPE

The objective of this project was to develop a series of standard abutment designs, a simple design methodology, and a series of design aids for the more commonly used substructure systems. These tools will assist Iowa County Engineers in the design and construction of low-volume road (LVR) bridge abutments. The following tasks were undertaken to meet the research objectives.

- Conduct a survey of the Iowa counties to determine current design practices and construction capabilities.
- Investigation of various LVR bridge abutments used by agencies outside of Iowa.
- Identify practical abutments for additional review.
- Develop a simple design methodology and series of standard abutment plans for the selected abutment systems.
- Create a series of standard abutment design aids.

Details on how these research objectives were achieved are presented in the following sections of this report.

1.3. REPORT SUMMARY

A similar version of this report is published as two volumes of the Iowa Department of Transportation (Iowa DOT) Project TR-486 final report [6, 7]. Volume I is similar to Chapters 1 through 9 and Appendices A and B of this report whereas Appendix C is similar to Volume II. Therefore, Appendix C has its own Table of Contents, chapters, appendices, etc. Chapters 1 through 9 and Appendices A and B of this report will be referred to as Volume I and Appendix C will be referred to as Volume II herein.

This report includes a survey of the Iowa County Engineers, the development of the abutment design methodology, standard designs, design aids, and a summary of additional research required. Many different sources of information were utilized in the development of the standard abutment plans and design aids. This includes technical articles, the websites of several state DOT's, plus the input of local Iowa Engineers. This input from the local Iowa Engineers was obtained from a survey distributed by the BEC to the Iowa County Engineers and from members of the Project Advisory Committee (PAC). The members of the PAC represented Iowa counties as well as the Iowa Department of Transportation (Iowa DOT).

Volume I includes the design methodology developed for this project. This includes the determination of gravity and lateral loads, performing the structural analysis, computing the system capacity, and performing various design requirement checks. A summary of research needed on alternative abutment systems (that are easy to construct, applicable in a wide range of situations, and are cost competitive) is also presented.

Volume II provides a set of LVR bridge abutment design aids and instructions on how to use them. All of the design aids and design equations are included in the appendices of Volume II. This includes; estimated gravity loads, driven pile foundation soils information chart, printouts from the foundation design template, generic standard abutment plans, and design methodology equations with selected figures.

In Volume II, three figures are provided to determine conservative dead and live load abutment reactions for various span lengths of some LVR bridge systems. A description of all input values required for using the foundation design template (FDT) along with recommendations for the optimization of a foundation design are presented. The instructions for using the standard abutment plans are also provided. It should be noted that by modifying the abutment bearing surface, superstructure systems described earlier in this report, plus essentially any other type of bridge superstructure system can be supported.

2. INPUT FROM IOWA ENGINEERS

The objective of this project was to create an easy-to-use design methodology and design aids to assist Iowa County Engineers in the development, design, and construction of various types of LVR bridge abutments. If this objective was to be accomplished, local engineers needed to be actively involved in the project (i.e., providing information, guidelines and recommendations to the project investigator). This included providing information on the design of the most common abutment systems, construction practices, and the county capabilities in these respective areas. This information was collected through a survey sent to the Iowa counties and from the PAC recommendations.

2.1. TR-486 SURVEY

Prior to this project, the design methodologies, construction practices, and capabilities of the Iowa counties were not entirely known. This included the details and type of bridge site investigations prior to design and construction, what percentage of counties designed and constructed their own abutments, what percentage hire a consultant or contractor for the substructure construction, the equipment and labor requirements for the construction the most common abutment types, and the common foundation element trends or patterns for various geographic locations throughout the state.

2.1.1. Objective and Scope of Survey

The objective of this survey was to obtain information relating to the common abutment designs and construction practices of Iowa counties. This information was collected to help guide other aspects of this project. One area of primary interest was the type and level of design work performed by the Iowa County Engineers for LVR bridge abutments. Specifically, it was desired to know if county engineering departments perform a majority of the design work in-house, or if private consultants are hired. Information relating to design methodologies and standard abutment designs that are commonly used as well as their limitations and applicability was also desired.

Another area of interest was related to the bridge foundation. This includes information on the type, quantity and typical depths of bridge foundation elements (i.e., steel and timber piles). Similarly, information regarding the common types of subsurface explorations and site tests was needed to fully understand typical county designs.

It was also desired to determine methods counties use in the construction of LVR bridge abutments. It was unknown if counties use county personnel for the construction of a typical LVR bridge or if private contractors are employed. Additionally, the type of equipment and the amount of labor required for the construction of a typical LVR bridge abutment was unknown.

In an attempt to answers these questions, TR-486 survey (included in Appendix A) was developed and sent to the Iowa County Engineers in the summer of 2003. This allowed the project investigator to obtain a better understanding of the county engineers design and construction capabilities and practices.

2.1.2. Survey Results and Summary

A detailed summary of the results of the survey is presented in Appendix B. The results in Appendix B are grouped according to the six Iowa DOT transportation districts. A brief summary of the complete survey results is presented below:

- Forty-six percent of counties (46 of 99 counties) completed and returned the survey.
- Seventeen percent (eight counties) stated that they use some type of standard abutment design; six counties sent drawings or plans.
- For the standard abutment designs that are used by Iowa counties, the following general limitations apply: single span lengths ranging from 20 to 90 ft, small or no skew angles, situations when shallow bedrock is not encountered, and when de-icing salts are not used.
- Twenty-six percent (12 counties) stated that they knew of other agencies with standard abutment plans. The other agencies listed include, other counties, Oden Enterprises, and the Iowa DOT. It should be noted that some counties that were mentioned stated that they did not use standard abutment plans.
- The equipment required for the construction of a typical LVR bridge abutment varied by county. Among the more common pieces of equipment mentioned were: cranes, vibrating and hammer pile drivers, excavators, and welders.
- The labor force required for construction of a standard abutment, when given in terms of man-hours, varied from 72 to 400 hours depending on the county. Some labor requirements were stated as: "four laborers" or "three to four workers, three to six weeks".
- Twenty-eight percent (13 counties) have their own bridge construction crew, 63 percent (29 counties) hire a contractor and nine percent (4 counties) use both alternatives.
- Fifty-six percent (26 counties) stated that some type of site investigation is performed before the installation of bridge foundation elements.
- Forty-five percent (21 counties) specifically stated that some type of subsurface exploration is performed and 13 percent (six counties) specifically cite that a SPT test is performed. No other specific soil test was mentioned.

- Sixty-five percent (30 counties) stated that steel H-piles are used at least some of the time whereas 33 percent (15 counties) use timber piles at least some of the time. Ten percent (5 counties total) use some type of reinforced concrete pile.
- The depth for steel H-piles ranged from 20 to 90 ft, depending on the county, with the most common depth being approximately 40 ft. The depth for timber piles ranged from 20 to 40 ft, depending on the county, with the most common depth being approximately 30 ft.

2.2. PROJECT ADVISORY COMMITTEE (PAC)

In addition to the results from the Iowa County Engineer's survey, the previously mentioned PAC was formed to provide additional information and guidance. Members of the PAC consisted of Brian Keierleber (County Engineer, Buchanan County), Mark Nahra (County Engineer, Delaware County), Tom Schoellen (Assistant County Engineer, Black Hawk County), and Dean Bierwagen (Methods Engineer, Iowa Department of Transportation). The PAC committee was created to provide the project investigator personal with professional input throughout the various stages of the project.

The PAC provided very valuable information relating to the scope of the project. In meetings with the PAC, it was decided that standard abutment designs should include roadway widths of 24 and 30 ft with span lengths ranging from 20 to 90 ft. It was also suggested that the standard abutment designs should accommodate different superstructure types such as the RRFC, BISB, pre-cast double-tee (PCDT), prestressed concrete girders (PSC), quad tee's, glued-laminated (glulam) timber girders, and slab bridges. Additionally, since 6 to 12 ft is a common range for the abutment backwall heights in Iowa, it was therefore decided to limit the designs to this range. The PAC noted that most Iowa counties primarily use steel and timber piles, thus these two materials should be the primary materials investigated for use in the abutment designs. Finally, members of the PAC stated that some type of computer based design aid would be very useful in assisting the county engineers in the design of the foundation elements. This design aid could be used if a particular bridge site did not fit the assumptions used in the abutment standards provided. This design aid would have to be easy to use and readily available. The operating system suggested by the members of the PAC was visual basic or an Excel spreadsheet.

After the initial scope of the project was defined, members of the PAC were frequently contacted about issues relating the design methodology and design aids. Issues such as the use of anchor systems, tiebacks, sheet piles, and lateral load analysis were all addressed. Additionally, members of the PAC provided guidance and suggestions on the practicality and format of the design aids being developed so they could be easily used by Iowa County Engineers.

3. LITERATURE REVIEW

A literature search was performed to collect information on standard abutment plans and design methodologies that are currently used for LVR bridge abutments. Several sources including: 1.) all state DOT websites, 2.) the Federal Highway Administration (FHWA), 3.) the Local Technology Assistance Program (LTAP) network, and 4.) the Transportation Research Information Services (TRIS) were used in the literature search.

The literature reviewed in this report is not intended to be all inclusive on the topic of LVR bridge abutments. This literature review focuses primarily on the information required and used to develop the standard abutment plans and a design methodology for this project. Some additional information, however, such as available standard abutment designs and alternative abutment systems are also included in this review.

3.1. ABUTMENT CLASSIFICATIONS

Abutments systems are generally classified as either integral or stub abutments. In an integral abutment, the superstructure is structurally connected to the substructure with a reinforced concrete end diaphragm, shear key, and/or reinforcing dowel rods. The structural connection subjects the piles to bending loads caused by thermally induced horizontal movements as well as the end rotation of the superstructure from live loads [8]. After a review of project survey results and the input of the FAC presented in Chapter 2, it was evident that integral abutments systems used in Iowa counties are based on the standard designs available through the Iowa DOT [9]. Thus, it was decided that there is already sufficient information available on integral abutments.

The structural connection to the superstructure associated with integral abutments is not used in a typical stub abutment system which is considered a simple support. As shown in Figures 3.1 and 3.2, a typical Iowa county stub abutment consists of a single, vertical row of either steel or timber piles. The pile cap typically consists of either steel channels connected to the pile heads (Figure 3.1) or a cast-in-place reinforced concrete cap (Figure 3.2). A backwall composed of either stacked horizontal timber planks or vertically driven sheet piles are placed behind the exposed piles to act as a retaining wall for the backfill soil. The total height of the backwall typically ranges from 6 to 12 ft, which includes the exposed pile length plus the combined depth of the roadway and superstructure. Some counties also use an anchor system to resist the horizontal substructure loadings. This system typically consists of a buried reinforced concrete anchor block (shown in Figures 3.1 and 3.2) that is connected to the abutment system with anchor rods and an abutment wale.

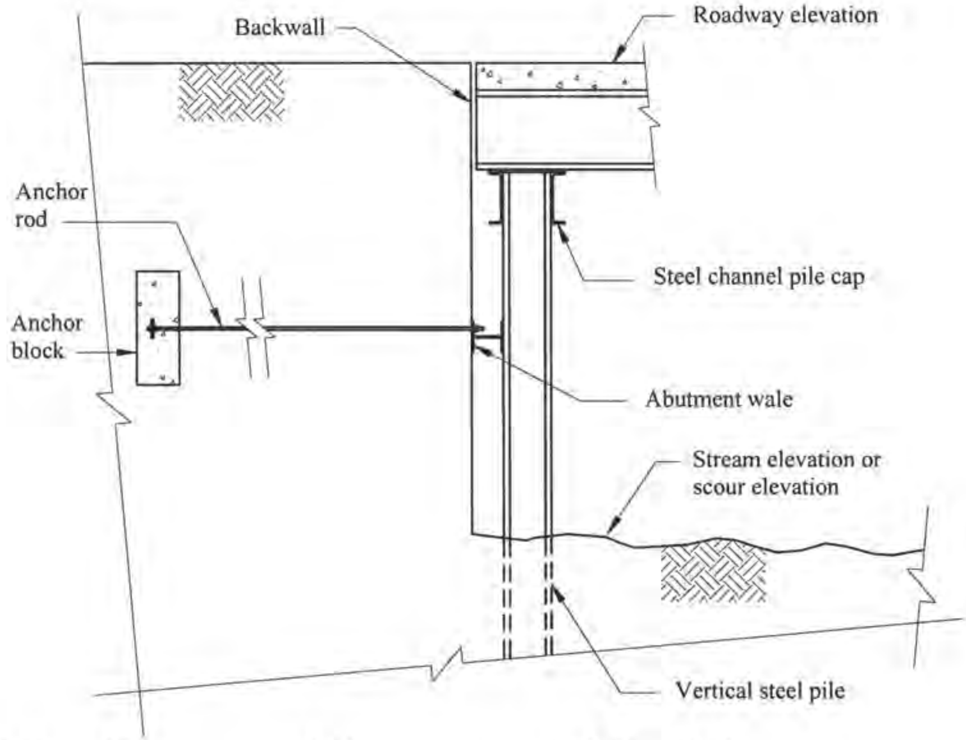


Figure 3.1. Typical Iowa county stub abutment using a steel channel pile cap.

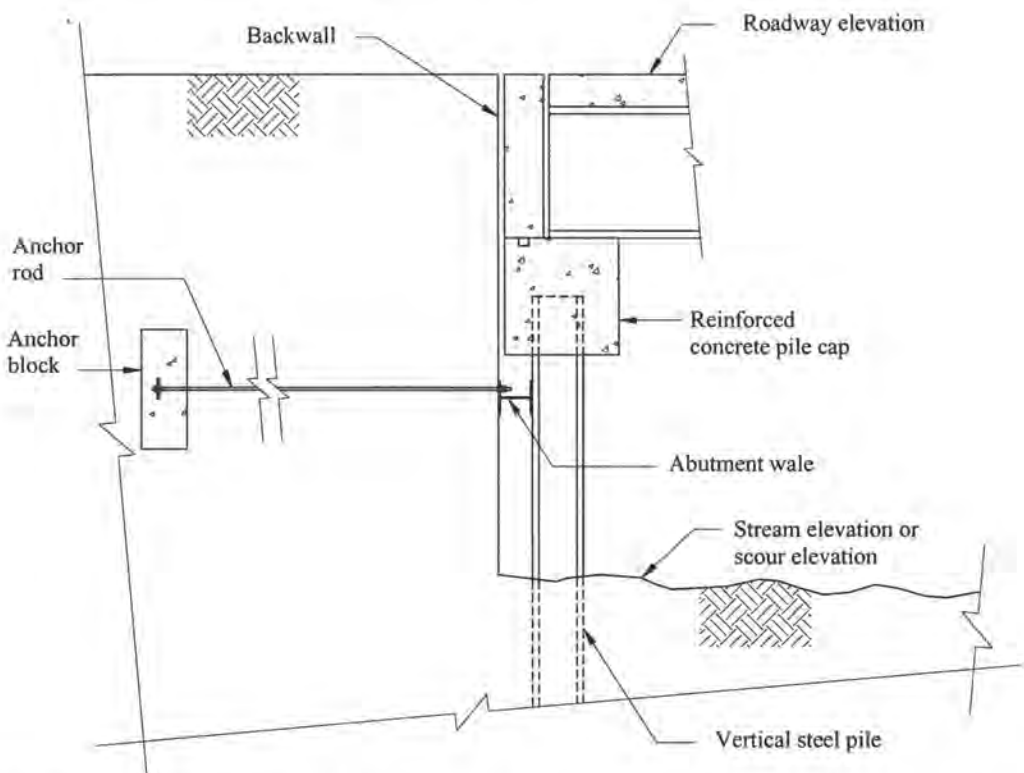


Figure 3.2. Typical Iowa county stub abutment using a cast-in place reinforced concrete pile cap.

Another stub abutment system used by several state DOT's is shown in Figure 3.3. This particular system has two rows of completely embedded piles with a cast-in-place reinforced concrete pile cap and backwall. The back row piles (i.e., farthest away from the stream edge) are vertically whereas the front row piles (i.e., nearest to the stream edge) are typically battered at a one horizontal to four vertical orientation [10, 11]. The battered piles contribute to the vertical bearing capacity in addition to resisting the horizontal loadings [12].

The literature search also revealed several additional economical systems that potentially can be used for LVR bridge abutments. This includes micropiles, geosynthetic reinforced soil structures, Geopier foundations, and sheet pile bridge abutments. These systems are well established in a particular geographic region or for a specific use, however none of them have been used as a bridge abutment system in Iowa. For this reason, these systems were not included with the standard abutment designs presented herein. However, a more detailed description of these systems is presented later in Chapter 5. In the future, these systems could be introduced into the Iowa transportation system on a trial basis and their performance evaluated (see Chapter 7).

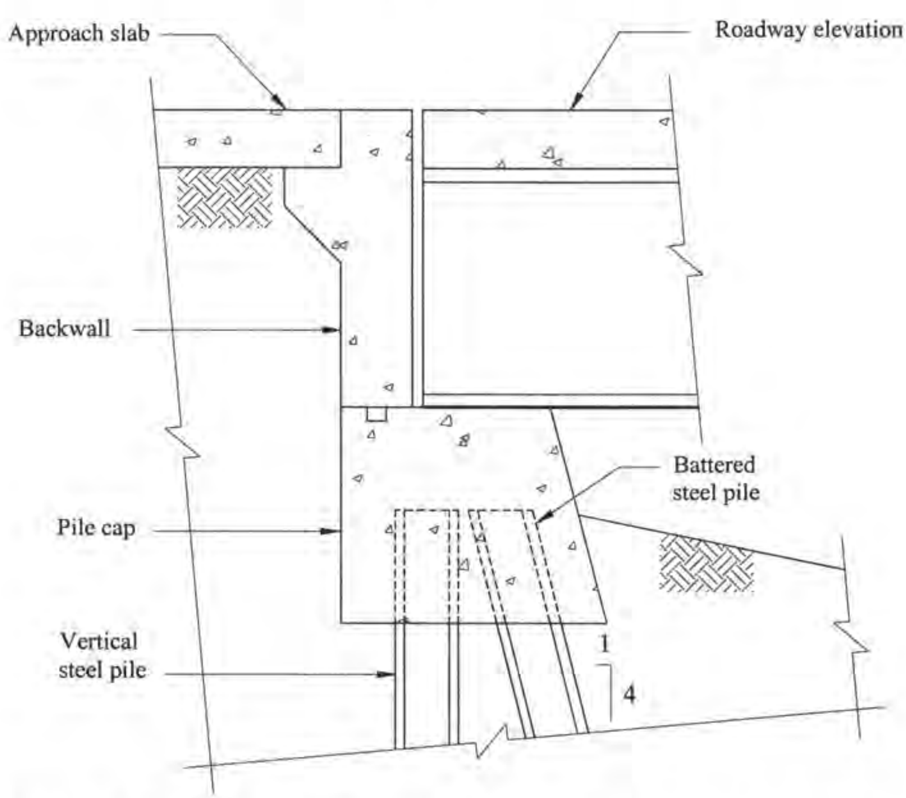


Figure 3.3. Example of a stub abutment system commonly used by many state DOT's [adapted from Iowa DOT standards designs].

3.2. AVAILABLE ABUTMENT DESIGN INFORMATION

An item of particular interest in this literature review was standard abutment plans and designs. The Iowa DOT developed a series of bridge standards for Iowa county roads [9]. These include standards for prestressed girder and slab bridges with either integral or stub abutments. For example, the H24S-87 standards provide complete superstructure and substructure details for a single span, prestressed concrete girder bridge with a roadway width of 24 ft and span lengths ranging from 30 to 80 ft. The substructure details are similar to those shown in Figure 3.2, this includes a single row of exposed timber piles, a timber plank backwall, and a cast-in-place reinforced concrete pile cap. Other Iowa DOT standard bridge designs include H24, H30, J24, and J30. These standards provide designs details for three-span prestressed girder and slab bridges with roadway widths of either 24 or 30 ft. The total bridge lengths range from 126 to 243 ft and 75 to 125 ft for the prestressed concrete girder and slab bridges standards, respectively. The substructure details consist of an integral abutment with a single row of vertical piles.

A review of all 50 state DOT websites revealed a number of different abutment standards available online. Most standards utilize fully embedded piles with either a cast-in-place or pre-cast reinforced concrete pile cap and backwall system. However, the Alabama DOT [13] gives the details for an abutment system similar to the Black Hawk County, Iowa stub abutment system shown in Figure 3.4. In this system, precast concrete panels are placed between adjacent piles to act as the backwall.

Figure 3.4. Stub abutment system with a precast concrete panel backwall [photo courtesy of Black Hawk County, Iowa].

Various state DOT websites, including Iowa, New York [14], Ohio [15], Oklahoma [16], Pennsylvania [11], and Texas [17] also provide abutment standards sheets on-line. Additionally, Pennsylvania and Oklahoma provide standards abutment designs specifically for LVR bridge abutments. The Pennsylvania DOT standard design sheets are in a generic format in which the engineer can calculate and then fill-in the necessary information (e.g. roadway width, etc.). The Oklahoma DOT LVR bridge abutment standards sheets are not generic, however standard sheets are available for different superstructure types, span lengths and skew angles.

The National Cooperative Highway Research Program (NCHRP) Synthesis 32-08: *Cost Effective Structures for Off-system Bridges* [18] provides a comprehensive summary of different organizations and government agencies with published bridge standard designs. For example, in the late 1970's and 1980's, the FHWA published bridge standards for concrete, steel and timber superstructures. Unfortunately, these bridge standards have not been updated to meet code changes. Other organizations such as the American Iron and Steel Institute, the Concrete Reinforcing Steel Institute, the U.S. Army Corps of Engineers, and the U.S. Navy Facilities Command have also published bridge standards that include substructure details.

3.3. LATERAL LOAD ANALYSIS TECHNIQUES

The foundation elements most commonly used for LVR bridge abutments in Iowa consist of vertical steel and timber piles as previously described. Two different methods for determining the pile behavior when subjected to lateral loads were reviewed for this project.

3.3.1. Non-Linear Analysis

The first lateral load analysis method is commonly known as the p-y method. This analysis technique utilizes a series of non-linear, horizontal springs to represent the soil reaction imparted on the pile when subjected to lateral loads. The pile is modeled as a string of elements with horizontal springs attached to the nodes as shown in Figure 3.5. The springs have stiffness properties similar to the surrounding soil. Each spring imparts a horizontal force on the pile that can be defined by the non-linear relationship of Equation 3.1 [12].

$$F = p\,y \tag{3.1}$$

where:

F = Spring force representing the soil reaction at the node location.

p = Non-linear soil stiffness that is a function of the lateral displacement.

y = Lateral displacement.

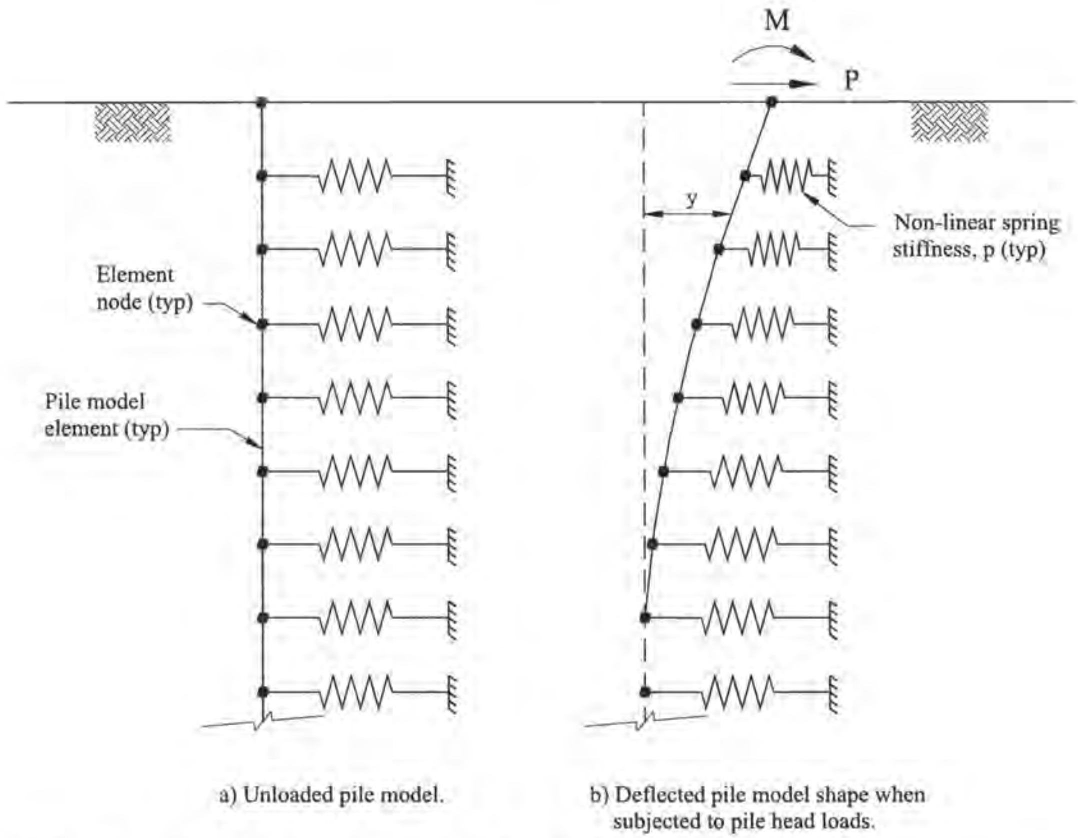


Figure 3.5. Pile model with non-linear springs [adapted from Bowles, 1996].

The magnitude of the applied soil stress has a significant influence on the soil stiffness. As the depth below the ground surface increases, the associated increase in vertical stress will induce an associated increase in the soil stiffness. Additionally, the lateral pile movement will also convey additional stresses on the soil. Because of the dependence on depth, different non-linear spring stiffness values are assigned to each spring in the pile model thus creating a statically indeterminate, non-linear system. Typically, empirical equations developed from lateral load tests are used to model the stiffness-deflection relationship of a particular soil [12]. A typical stiffness-deflection relationship is shown in Figure 3.6.

3.3.2. Linear Analysis

The second lateral load analysis method was developed by Broms [19, 20]. This method considers a sufficiently long pile, fixed at a calculated depth below ground. By assuming a point of fixity, the pile can be analyzed as a cantilever structure with appropriate boundary conditions and external loadings. The calculated depth to fixity is a function of the soil properties, pile width, lateral loadings and pile head boundary conditions. The pile moment and deflection can be determined using structural analysis techniques. The depth to fixity for a pile in a cohesive soil is presented in

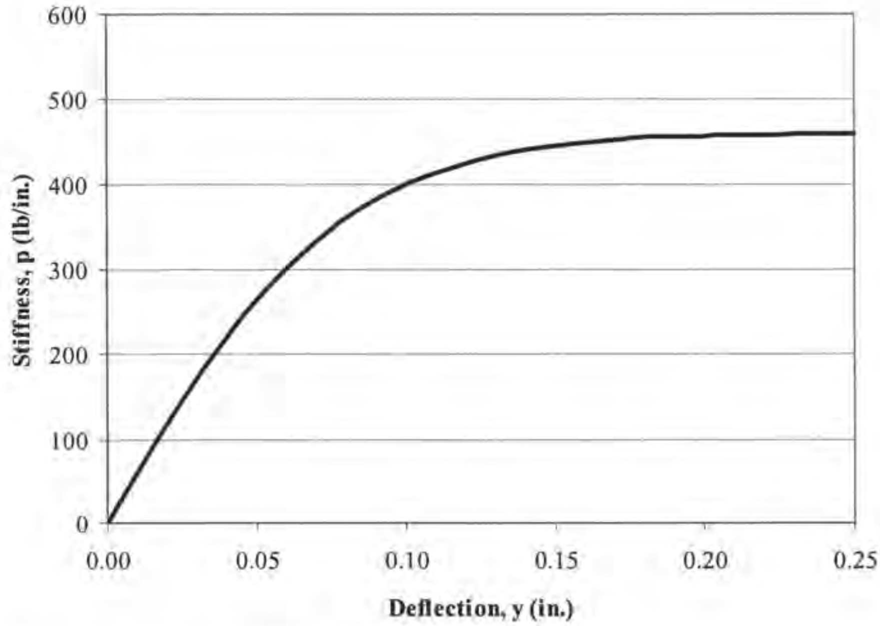


Figure 3.6. Example of a typical stiffness-deflection (p-y) curve.

Equation 3.2. The general deflected shape, passive soil reaction, and moment diagram for a pile in a cohesive soil is shown in Figure 3.7.

$$L = 1.5\,B + f \qquad (3.2)$$

where:

B = Pile width parallel to the plane of bending.

f = Length of pile required to develop the passive soil reaction to oppose the above ground lateral pile loads (determined using Equation 3.3).

L = Depth to fixity below ground level.

The first term in Equation 3.2 represents the distance in which no passive soil reaction acts on the pile as shown in Figure 3.7. The second term represents the length of pile required to develop the passive soil reaction to oppose the above ground lateral pile loads which is determined using Equation 3.3. The length of pile determined using Equation 3.3 is used to obtain the pile moment at the point of fixity (Equation 3.4).

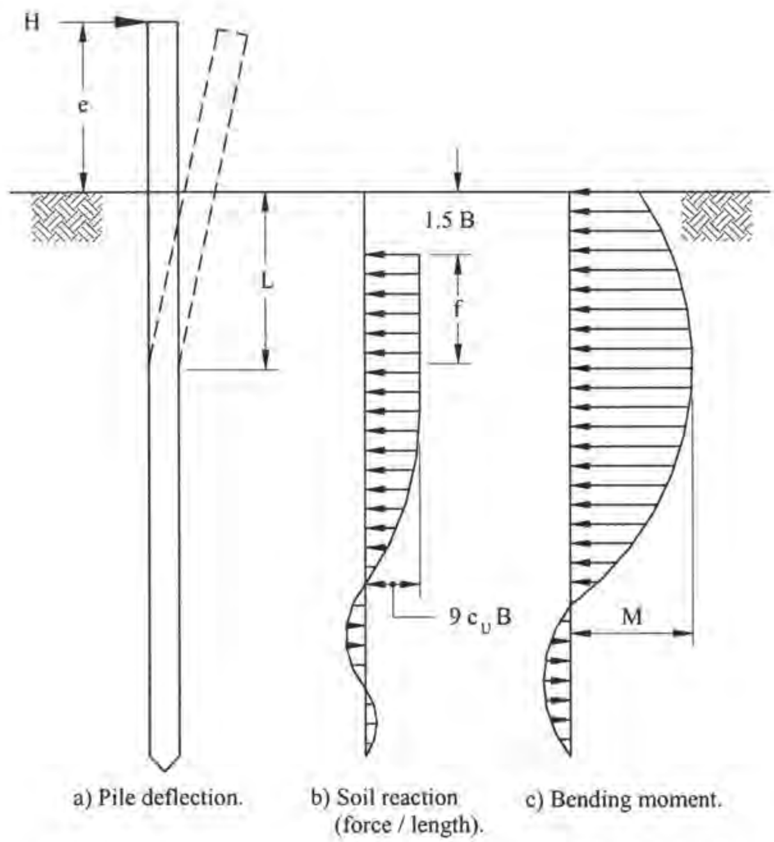


Figure 3.7. Behavior of a laterally loaded pile in a cohesive soil [adapted from Broms, March 1964].

$$f = \frac{H}{9\,c_U\,B} \tag{3.3}$$

where:

- B = Pile width parallel to the plane of bending.
- c_U = Undrained shear strength of the soil.
- f = Length of pile required to develop the passive soil reaction to oppose the above ground lateral pile loads.
- H = Total magnitude of the above ground lateral pile loads.

$$M = H\left(e + 1.5\,B + 0.5\,f\right) \tag{3.4}$$

where:

- B = Pile width parallel to the plane of bending.
- e = Distance above ground level to the centroid of the lateral pile loads.

f = Length of pile required to develop the passive soil reaction to oppose the above ground lateral pile loads (determined using Equation 3.3).

H = Total magnitude of the above ground lateral pile loads.

M = Pile moment at the point of fixity.

The general deflected shape, passive soil reaction, and moment diagram for a long pile in a cohesionless soil is shown in Figure 3.8. For cohesionless soils, the soil friction angle is the required soil shear strength parameter. The depth to pile fixity is calculated using Equation 3.5. This equation represents the length of pile required to develop the necessary passive soil reaction to oppose the above ground lateral pile loads. The depth to pile fixity is used to determine the pile moment at the point of pile fixity (Equation 3.6).

$$f = 0.82\sqrt{\frac{H}{\gamma B K_P}} \tag{3.5}$$

where:

B = Pile width parallel to the plane of bending.

f = Depth to fixity below ground level and length of pile required to develop the passive soil reaction to oppose the above ground lateral loads.

H = Total magnitude of the above ground lateral pile loads.

$K_P = \dfrac{1+\sin\phi}{1-\sin\phi}$ = Rankine passive earth pressure coefficient.

γ = Soil unit weight.

ϕ = Soil friction angle.

$$M = H\left(e + 0.67\,f\right) \tag{3.6}$$

where:

e = Distance above ground level to the centroid of the lateral pile loads.

f = Depth to fixity below ground level (determined using Equation 3.5).

H = Total magnitude of the above ground lateral pile loads.

M = Moment at the point of fixity.

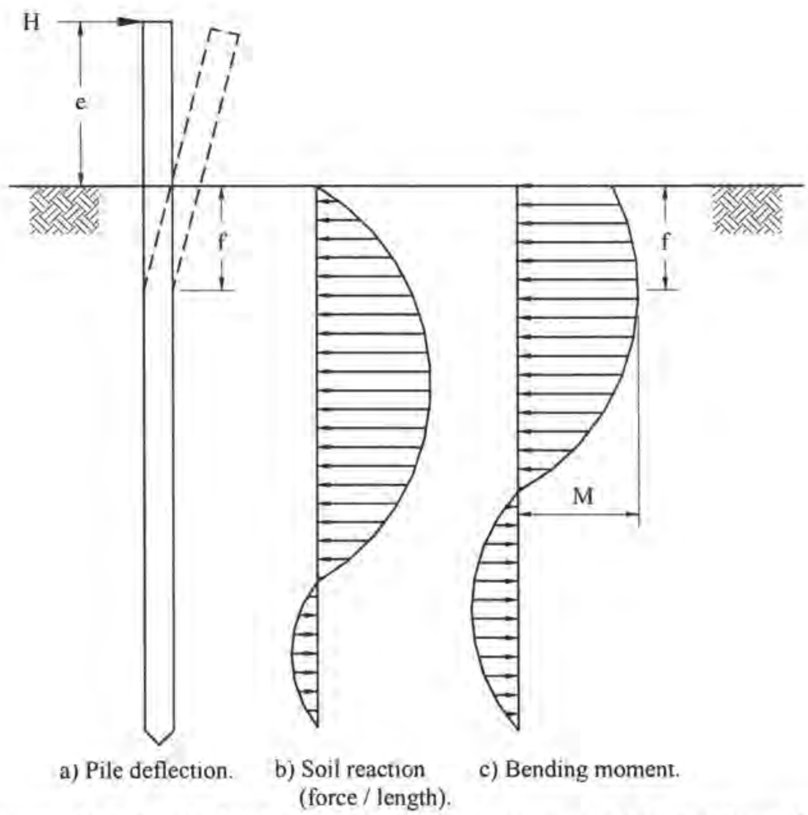


Figure 3.8. Behavior of a laterally loaded pile in a cohesionless soil [adapted from Broms, May 1964].

3.3.3. Lateral Load Analysis Comparison

The computer software, LPILE Plus v.4.0 utilizes the non-linear analysis technique and was used to investigate the maximum pile moment for different soil conditions when subjected to lateral loads. A significant limitation of LPILE is that all above ground lateral pile loads must be applied at the pile head. Therefore the lateral loadings previously described were resolved to this location as shown in Figure 3.9. An example of the lateral pile loading from the active earth pressure acting on the backwall is shown in Figure 3.9a (discussed later in Chapter 4) and the equivalent point load is provided in Figure 3.9b. When the point load is moved to the pile head, a moment is applied as shown in Figure 3.9c to produce the same pile moment at point 'A' (i.e., at point A the moments, M_1 and M_2 for Figures 3.9b and 3.9c respectively, are equal to P (z – e)).

Once the lateral loads were established, the various soil properties were defined. Initially, eight different homogenous soil conditions were investigated including two cohesive soils with SPT blow counts of 2 and 25 plus six cohesionless soils with blow counts ranging from 6 to 40. These soils were selected from Table 1.2 of the Iowa DOT Foundation Soils Information Chart (Iowa DOT

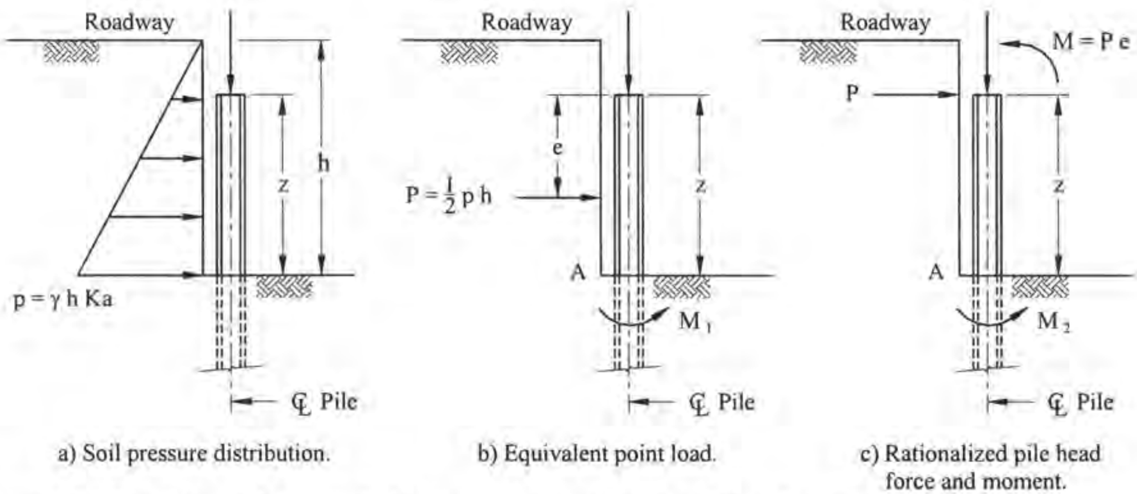


Figure 3.9. Resolving a lateral pile loading to an equivalent pile head point load and moment.

FSIC) [21] which is presented in Appendix B of Volume II. This table, which is described later (in Chapter 4), provides estimates of the allowable friction and end bearing values for various soils based on the SPT blow count.

The eight soil conditions previously stated can be classified into one of three categories for the LPILE, soft or stiff cohesive soils and cohesionless soils. For soft cohesive soils, the undrained shear strength and the soil strain value corresponding to one-half the maximum principal stress difference (ε_{50}) are required for LPILE in addition to the soil unit weight. Terzaghi and Peck [22] reports one of the more commonly used correlations between the SPT blow count and the undrained shear strength and is given by Equation 3.7. This relationship was selected because the Iowa DOT FSIC also correlates the SPT blow count to soil bearing properties. Since this correlation can be unreliable for all in-situ conditions, it is recommended that the undrained shear strength be determined by testing soil samples from the bridge site. Estimated ε_{50} values used in this study were obtained from the LPILE Technical Manual [23]. The values used for these soil parameters in LPILE, in addition to other soil parameters described later in this section, are provided in Table 3.1.

$$c_U = 0.06\,N\,P_{ATM} \tag{3.7}$$

where:

c_U = Undrained shear strength.

N = SPT blow count.

P_{ATM} = Atmospheric pressure.

Table 3.1. Summary of the soil properties used in LPILE.

SPT Blow Count	Soil Type	c_U	ϕ	ε_{50} *	k **	γ
N	-	(psf)	(degrees)	(in. per in.)	(lb per in.^{3})	(lb per ft^3)
2	soft cohesive	253	-	0.0201	-	115
6	cohesionless	-	28.6	-	100	115
12	cohesionless	-	30.7	-	150	115
20	cohesionless	-	33.3	-	200	115
40	cohesionless	-	38.5	-	500	115
25	cohesionless	-	34.8	-	250	115
25	stiff cohesive	3,175	-	0.0040	2,000	115
35	cohesionless	-	37.4	-	400	115

* - Obtained from Table 3.2 or 3.4 of the LPILE Technical Manual for soft and stiff cohesive soils, respectively.

** - Obtained from Table 3.3 or Figure 3.29 of the LPILE Technical Manual stiff cohesive soils and cohesionless soils, respectivley.

In addition, the undrained shear strength and ε_{50} values for stiff cohesive soils modeled in LPILE also require the modulus of subgrade reaction. The modulus of subgrade reaction is a relationship between the applied soil pressure and corresponding deflection and is commonly used for the structural analysis of foundation elements [12]. The LPILE Technical Manual was used to estimate the modulus of subgrade reaction based on the undrained shear strength of the stiff cohesive soil. As before, Equation 3.7 and the LPILE Technical Manual were both used to determine the undrained shear strength and the value for ε_{50}, respectively.

For cohesionless soils, LPILE requires the unit weight of the soil plus two additional soil properties including the modulus of subgrade reaction estimated from the LPILE Technical Manual and the soil friction angle. Peck et al. [24] reports a correlation used to obtain the friction angle based on the SPT blow count presented in Equation 3.8. It is recommended that the soil friction angle be determined from tests on soil samples from the bridge site.

$$\phi = 53.881 - \left(27.6043 * e^{-0.0147\,N}\right) \tag{3.8}$$

where:

N = SPT blow count.

ϕ = Soil friction angle.

The linear analysis technique reported by Broms [19, 20] was also used to determine the maximum moment of laterally loaded piles for different soil conditions. The undrained shear strength and soil friction angle are required for cohesive and cohesionless soils, respectively. The SPT blow count correlations defined by Equations 3.7 and 3.8 can also be used for this analysis method. As previously noted, the depth to fixity and the corresponding pile moment is determined using Equations 3.2 through 3.6 for the respective soil types.

A comparison of the two lateral load analysis techniques reveals advantages for both methods. The non-linear method can be used for more complex soil conditions such as a non-homogenous soil profile. It also provides a more accurate representation of the moment distribution along the length of the pile. However, specialized geotechnical software such as LPILE is needed to perform this analysis.

The second linear method does not account for the redistribution of pile loads below the point of fixity. Additionally, the soil pressure distributions used to determine the depth to fixity and the shape of the soil reactions were developed in the 1960's and may not be entirely accurate based on the non-linear soil load-deflection response shown in Figure 3.6. However, once the shape of the soil reactions is established, the pile deflection and moment along the length of the pile above the point of fixity can easily be determined. This analysis technique can also be incorporated into commonly available spreadsheet software.

Although the non-linear and linear methods use different assumptions and modeling techniques, they produce comparable maximum pile bending moments for different soil types and lateral loadings. The linear method is more conservative for stiff cohesive soils when compared to the non-linear method. The relationship between the maximum pile moment and backwall height is shown in Figure 3.10 for piles in stiff cohesive soil (SPT blow count of N = 25) spaced on 2 ft – 8 in. centers. Figure 3.10 reveals that as the magnitude of the lateral pile loads decrease (i.e., the backwall height decreases), the maximum pile moments obtained from the linear method are more conservative by 15 percent. As the magnitudes of the lateral loads increase (i.e., the backwall height increases), the maximum pile moments obtained using the linear method are more conservative by approximately seven percent.

Unlike stiff cohesive soils, the linear method produces less conservative maximum pile moment values in soft cohesive soils when compared to the non-linear method. The relationship between the maximum pile moment and backwall height is shown in Figure 3.11 for piles in soft cohesive soil (SPT blow count of N = 2) also spaced on 2 ft – 8 in. centers. As the magnitude of the lateral loads decreases, the difference between the two analysis methods increases. In this case, the

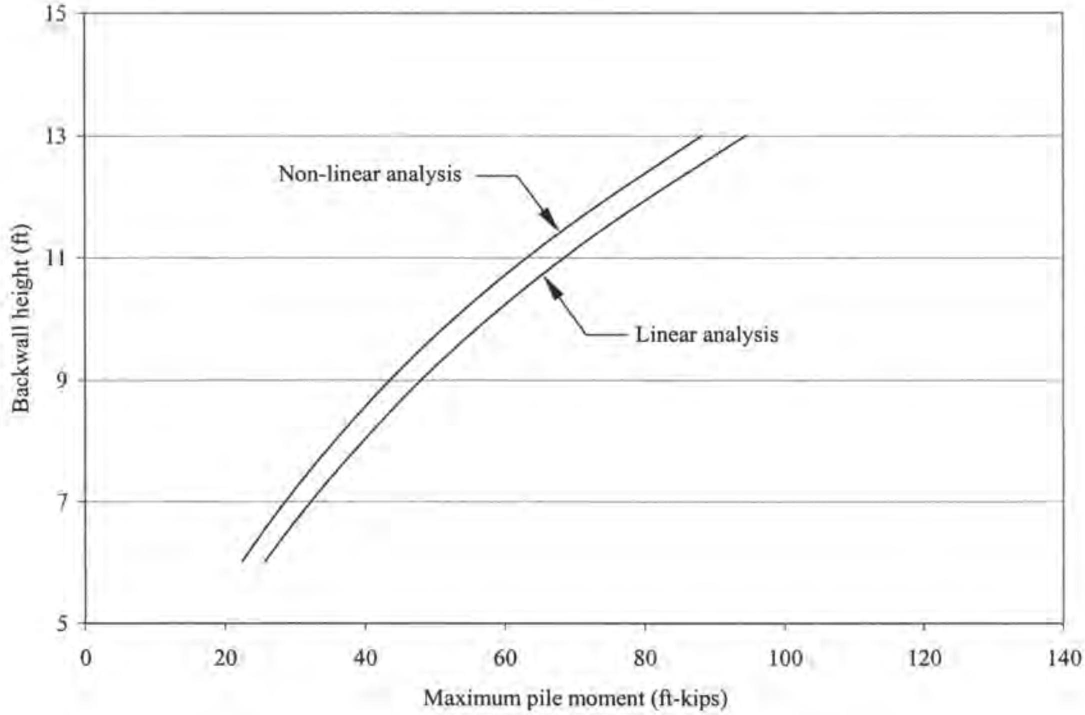


Figure 3.10. Maximum pile moment vs. backwall height for piles spaced on 2 ft – 8 in. centers in stiff cohesive soil (SPT blow count of N = 25).

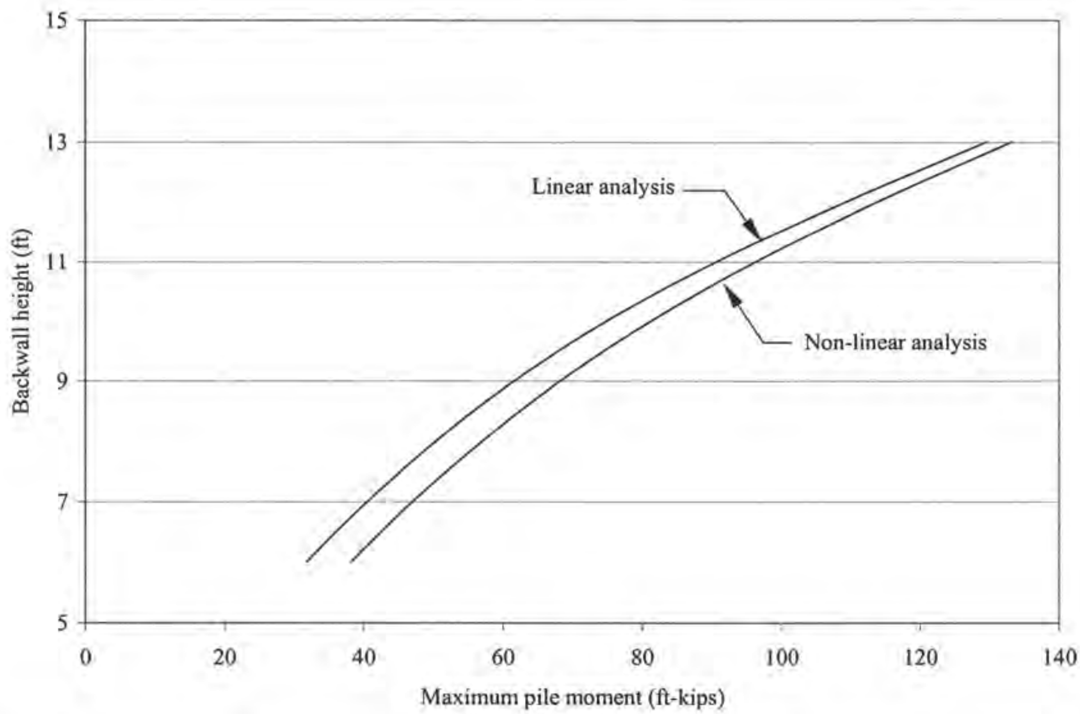


Figure 3.11. Maximum pile moment vs. backwall height for piles spaced on 2 ft – 8 in. centers in soft cohesive soil (SPT blow count of N = 2).

linear method is less conservative by about 20 percent for lower magnitude loadings. As the magnitude of the lateral loads increases, the two methods converge to within three percent.

Finally, the maximum pile moment values in cohesionless soils obtained from the linear method are slightly more conservative than the non-linear results. The relationship between the maximum pile moment and backwall height is shown in Figure 3.12 for piles in cohesionless soil (SPT blow count of N = 25) spaced on 2 ft – 8 in. centers. This conservative difference ranges from zero to three percent and does not vary significantly as the magnitude of the lateral pile loads change.

As previously stated, the linear method was less conservative for soft cohesive soils by up to 20 percent. However, given the assumptions used for the development of this design methodology, the general similarity in results when compared to the non-linear method, and the reduced computational requirements, the linear method, presented by Broms [19, 20], is used in the design methodology for LVR bridge abutments developed in this investigation.

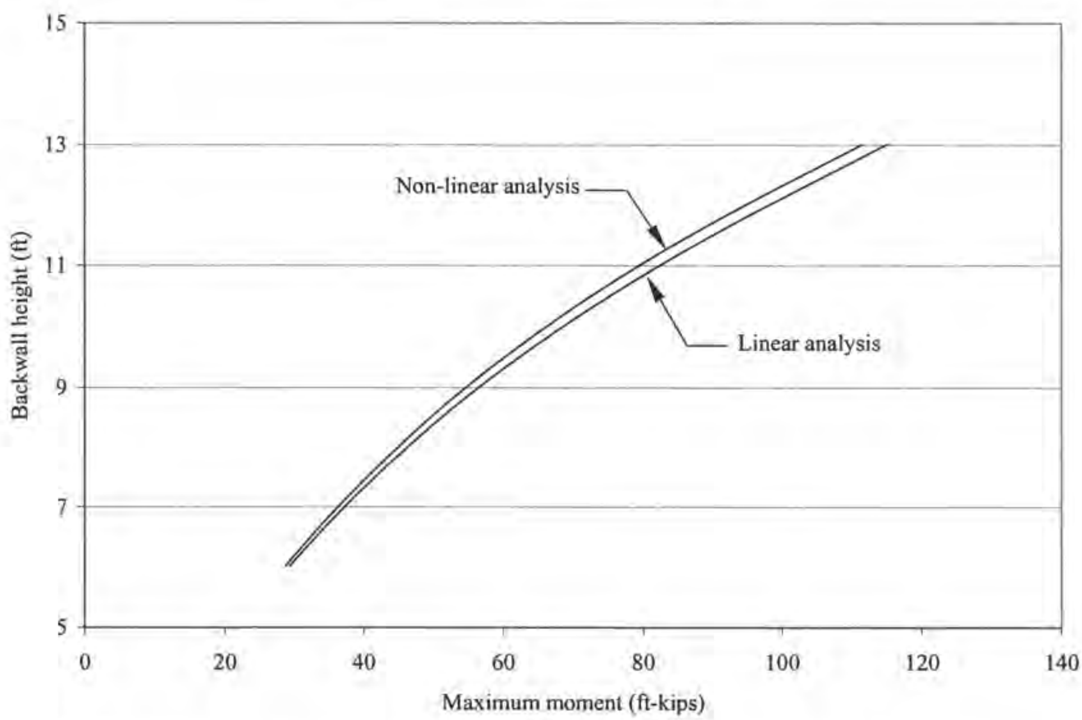


Figure 3.12. Maximum pile moment vs. backwall height for piles spaced on 2 ft – 8 in. centers in cohesionless soil (SPT blow count of N = 25).

4. DESIGN METHODOLOGY

In this chapter, a design methodology is developed for the foundation elements most commonly used for LVR bridge abutments in Iowa. This includes determination of substructure loads, structural analyses, determination of the pile and anchor system capacities, and design verification. An overview of additional substructure elements such as pile caps, abutment wales, and backwalls is also presented. A graphical representation of the design methodology is shown in Figure 4.1.

4.1. DESIGN LOADS

Once the basic substructure configuration is established (i.e., the number of piles, the lateral restraint system, and the corresponding system properties), the substructure loads, must be identified. This step is denoted as Part 'A' in Figure 4.1. Gravity loads include bridge live loads and dead loads due to the superstructure and substructure self weight. Lateral loadings are imparted to the bridge substructure by active and passive soil pressures in addition to lateral braking and wind loads transmitted through the bridge bearings.

4.1.1. Gravity Loads

The identification of substructure gravity loads includes the self-weight of the bridge roadway surface, superstructure, and substructure in addition to bridge live loads. The total abutment reaction is obviously equal to the sum of the dead and live load reactions.

4.1.1.1. DEAD LOAD

Conservative dead load abutment reactions for PCDT, PSC, quad tee, glulam, and slab bridge systems are shown in Figures 4.2 and 4.3 for 24 and 30 ft roadway widths, respectively. It should be noted that the PCDT dead load abutment reactions can also be used for steel girder superstructures. These estimated abutment reactions are based on published standard design sheets for the respective superstructure systems and include the self weight of both the superstructure and substructure. More accurate and potentially smaller dead load abutment reactions can be calculated by using site-specific bridge information. The dead load abutment reactions for other standard superstructure systems such as the RRFC and BISB systems are not included since there are numerous cross sections available which results in different self weights.

A number of conservative assumptions, applicable to all superstructure systems previously listed, were used to estimate the dead load abutment reactions shown in Figures 4.2 and 4.3. For all superstructure systems, a 20 psf future wearing surface was assumed in addition to two thrie-beam rails, with a conservatively estimated weight of 50 plf per rail, were assumed for all superstructure

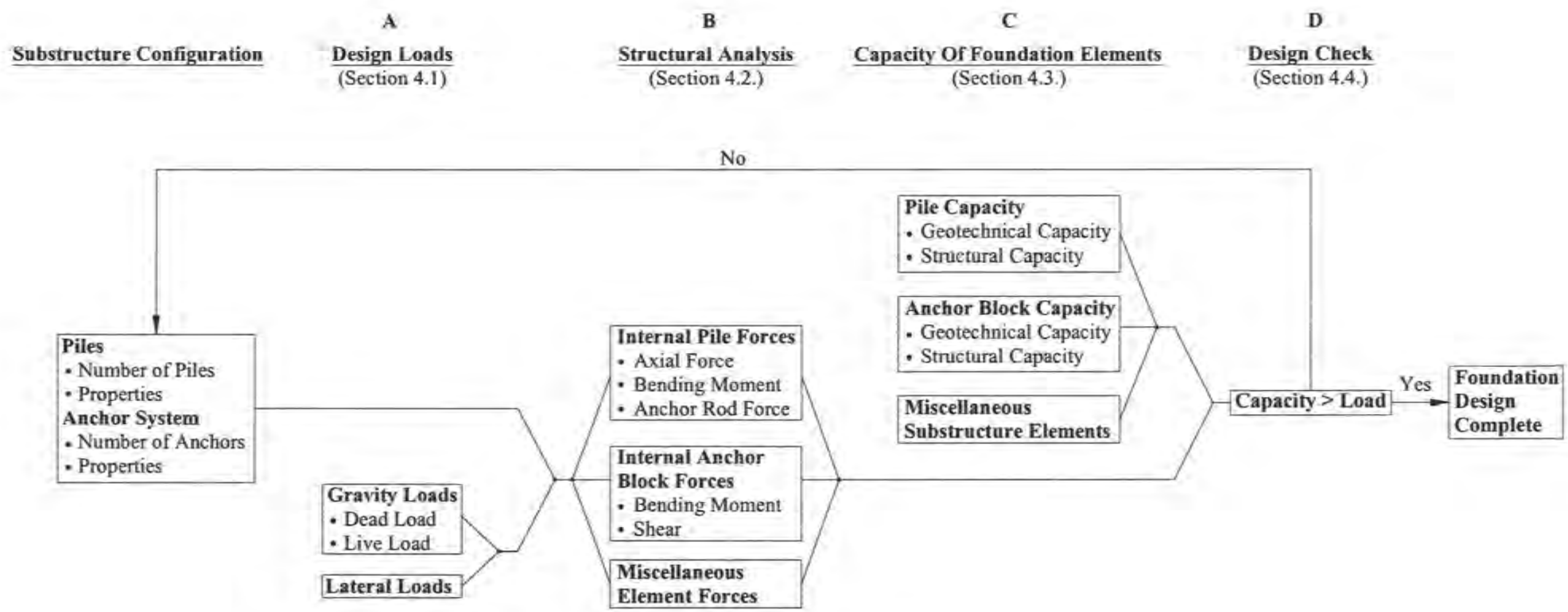


Figure 4.1. Graphical representation of the design methodology for a LVR bridge abutment.

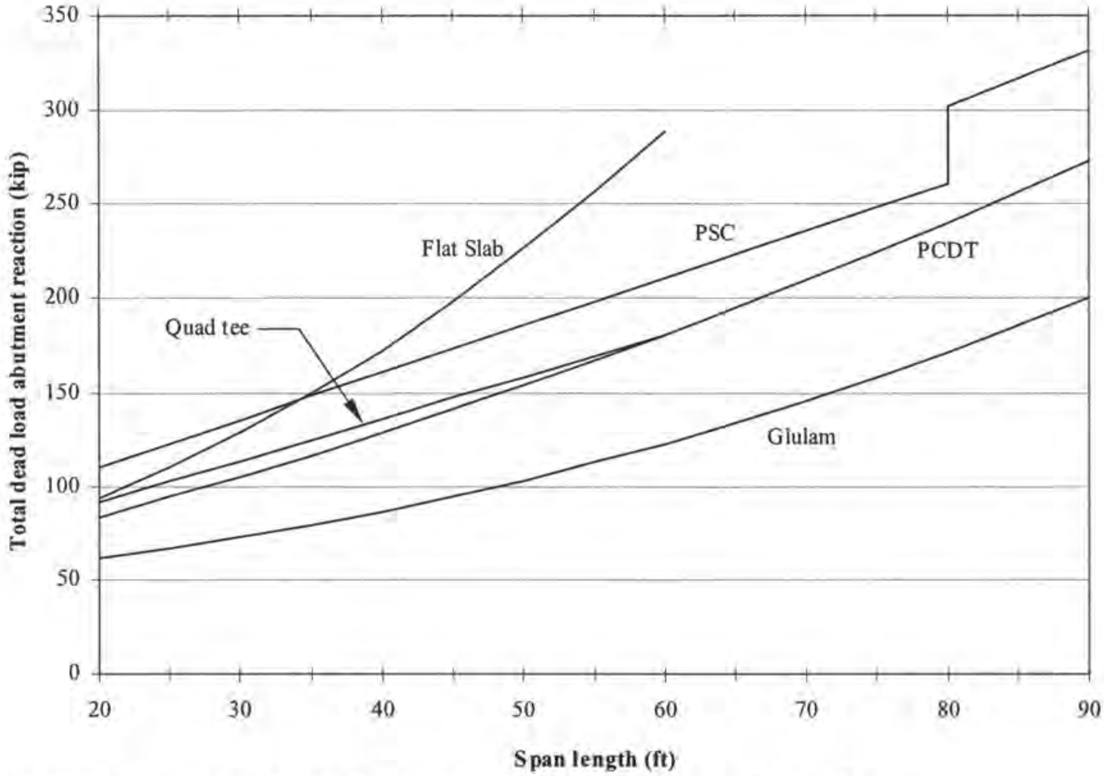


Figure 4.2. Estimated dead load abutment reactions for a 24 ft roadway width.

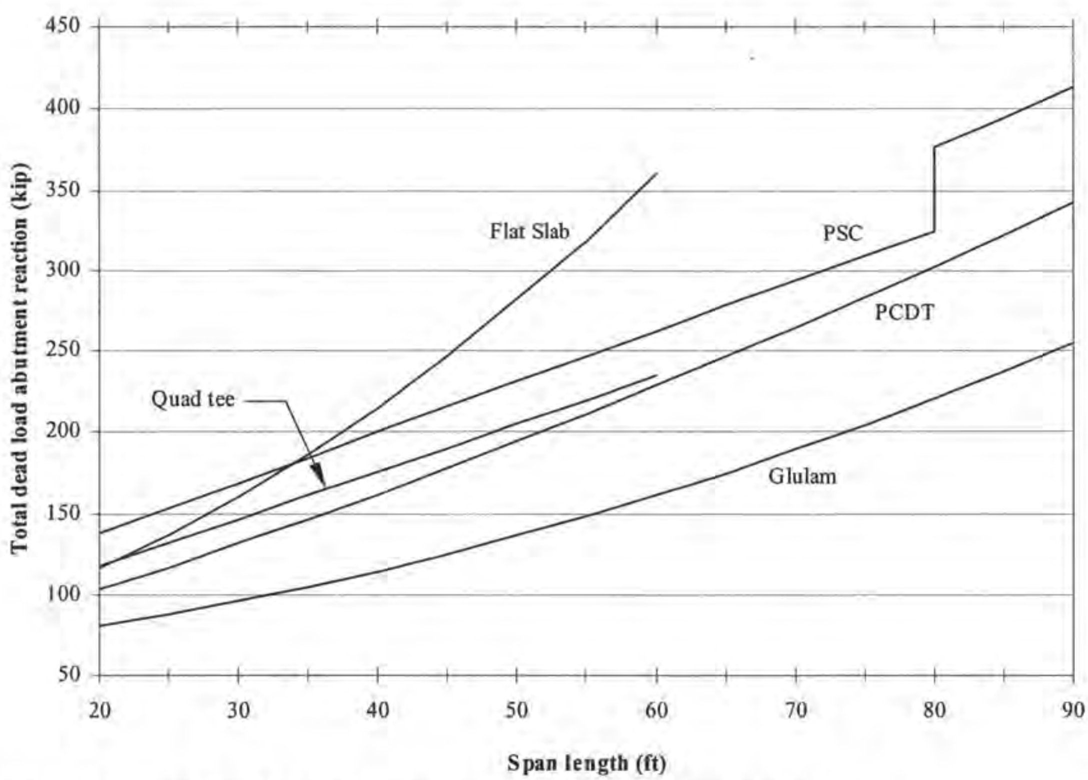


Figure 4.3. Estimated dead load abutment reactions for a 30 ft roadway width.

systems. Many LVR bridge systems use a concrete end diaphragm that acts as soil retaining wall above the pile cap. If the beams are encased in this end diaphragm there will be some end restraint and behavior similar to an integral abutment will occur. This type of connection is not included in this design methodology, however the weight of this wall was included. The estimated substructure dead load includes a three foot by three foot concrete pile cap with a length equal to the roadway width. Additionally, all estimated dead load abutment reactions were increased by five percent because standards for non-specific bridge sites were used.

A list of the assumptions used to estimate the dead load of the superstructure systems shown in Figures 4.2 and 4.3 follows:

<u>Glulam Girders</u>

- United States Department of Agriculture Standard Plans for Timber Bridge Superstructures (2001) [25] were used as a guide for the deck and girder self weight calculations.
- Since standard design sheets for a 30 ft roadway width were not available, a 32 ft roadway width was used (Figure 4.3).

<u>PSC</u>

- Iowa DOT H24S-87 standard design sheets [9] for a 24 ft, single span PSC system were used as a guide for the slab and girder self weight calculations.
- Five girders were used for the 30 ft roadway width (Figure 4.3).
- The Iowa DOT LXC standard girder section [9] was used for span lengths ranging from 20 to 80 ft.
- The Iowa DOT LXD standard girder section [26] was used for span lengths ranging from 80 to 90 ft.

<u>PCDT</u>

- PCDT standard design sheets published in Iowa DOT Project TR-410 [27] were used as a guide for the slab and girder self weight calculations.

<u>Quad Tee</u>

- The Cretex Concrete Products Midwest, Inc. (formerly known as Iowa Concrete Products Company) standard quad tee section [28] was used to estimate the superstructure self weight.
- Six and eight quad tee sections were used for the 24 and 30 ft roadway widths, respectively.

Slab Bridge

- Iowa DOT J24-87 standard design sheets [9] for a 24 ft, three span slab bridge were used as a guide for superstructure self weight calculations.
- The center span length to slab depth ratios of the J24-87 standard design sheets were used to estimate the slab depths for all applicable span lengths.

4.1.1.2. LIVE LOAD

The live load abutment reaction is computed using the 1996 American Association of State Highway Transportation Officials Standard Specifications for Highway Bridges, Sixteenth Edition (AASHTO) [29] HS20-44 design truck. Additional live loads such as the AASHTO lane load and Iowa legal loads were also investigated, however the HS20-44 truck controls for all span lengths defined for the scope of this project (i.e., between 20 and 90 ft). The maximum simple span loading occurs when the back axle is placed directly over the centerline of the piles with the front and middle axles on the bridge. The live load abutment reactions for two, 10 ft wide design traffic lanes without impact are presented in Figure 4.4. These values can be proportioned for a different number of design traffic lanes depending on the roadway width. Additionally, AASHTO defines a lane reduction factor that accounts for the probability of multiple lane loadings. If the number of 10 ft design lanes is equal

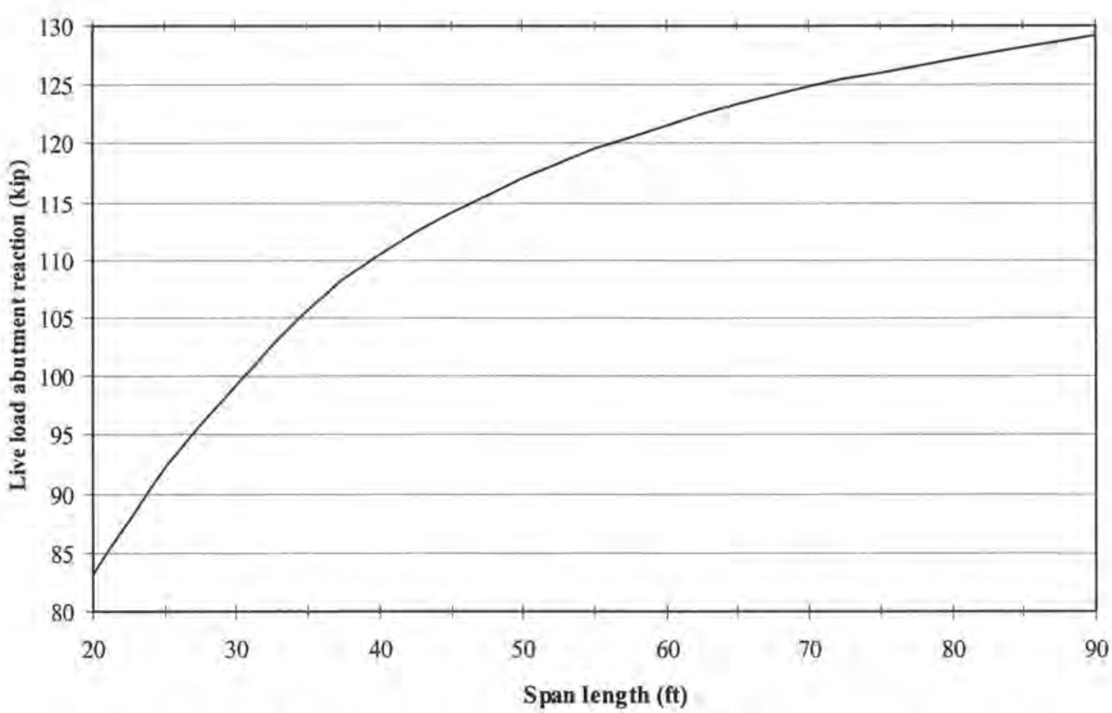


Figure 4.4. Maximum live load abutment reaction without impact for two, 10 ft design lanes.

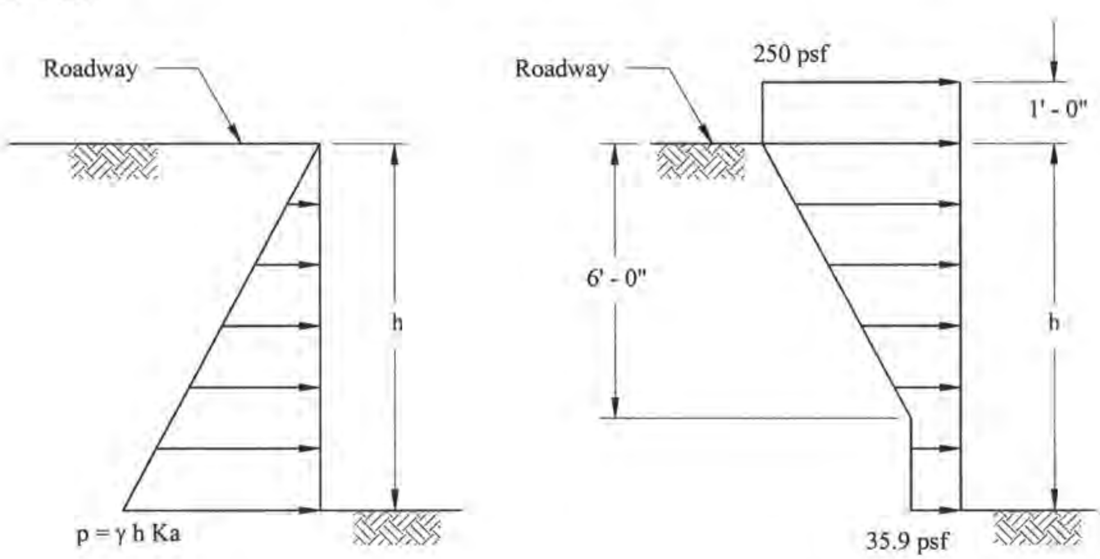
to three, then 90 percent of the live load is applied. If four or more design lanes are used, the live load is reduced to 75 percent. Live load impact should not be included in the design of substructure elements embedded in soil (i.e., piles and the anchor system) as cited in Section 6.5 of the Iowa DOT BDM.

4.1.2. Lateral Loads

The substructure systems commonly used by Iowa counties are required to resist lateral as well as gravity loads. One type of lateral loading results from soil pressures acting on the substructure. Additional superstructure lateral forces are transmitted to the substructure through the bridge bearings.

The Iowa DOT Bridge Design Manual (Iowa DOT BDM) [10] defines two different horizontal soil pressures for bridge substructures as shown in Figure 4.5. The active soil pressure attributed to the permanent loading of the backfill soil is shown in Figure 4.5a. The magnitude of this soil pressure is determined as a function of backwall height, h, using Equation 4.1. The Iowa DOT BDM cites values of 125 pcf and 33.7 degrees for the unit weight and friction angle, respectively.

The second Iowa DOT soil pressure distribution, presented in Figure 4.5b, is used to represent a live load on the approaching roadway. This live load is modeled as an equivalent soil surcharge equal to two feet with a unit weight of 125 pcf, thus resulting in the 250 psf shown in Figure 4.5b.



a) Active soil pressure distribution.

b) Equivalent live load surcharge.

Figure 4.5. Lateral soil pressure distributions [adapted from the Iowa DOT BDM, 2004].

$$p = \gamma h K_a \tag{4.1}$$

where:

h = Backwall height.

$K_a = \dfrac{1-\sin\phi}{1+\sin\phi}$ = Rankine active earth pressure coefficient.

p = Dead load active earth pressure.

ϕ = Soil friction angle.

γ = Soil unit weight.

As shown in Figure 4.5, the magnitudes of the lateral soil loadings are proportional to the backwall height. Because the scour on the streamside face of the backwall can wash away soil and effectively increase the backwall height, an estimated depth of scour should be considered if the geological and hydraulic conditions in the vicinity of the bridge site are conducive to this type of behavior.

Other lateral bridge loadings such as longitudinal wind forces, transverse wind forces, and a longitudinal braking force are also listed in the Iowa DOT BDM. The longitudinal braking force is equal to five percent of the AASHTO lane gravity loading multiplied by the number of 10 ft design lanes. One type of wind load consists of a 50 psf pressure that acts on the superstructure, roadway and barrier rail elevation surface area and acts perpendicular to the flow of traffic. A second wind load, also acting perpendicular to the flow of traffic, consists of a 100 plf line load that represents a wind force acting on the bridge live load. The load groups cited in Section 6.6 of the Iowa DOT BDM are used to determine the maximum loading effects for the various combinations of gravity and lateral loadings.

4.2. STRUCTURAL ANALYSIS

Once the substructure loads have been defined, the structural analyses of the various systems can be performed to determine the internal element design forces. These forces include the pile axial load and bending moment, anchor rod forces, and the anchor block shear and bending moment. This step is denoted as Part 'B' in Figure 4.1.

4.2.1. Internal Pile Forces

4.2.1.1. AXIAL PILE FORCE

As previously discussed, the abutment reaction is the sum of the dead and live load reactions which are used to determine the individual axial pile load. The axial pile loads (i.e., the load each pile much resist) are a function of the total number of piles and their spacing plus the superstructure bearing points. Thus, a nominal axial pile factor was developed for different situations. Various combinations of superstructure systems and pile spacings were analyzed by creating a series of pile cap models in a structural analysis software program. The pile cap was modeled as a continuous beam with the conservative assumption of pinned supports representing the piles. The loading consisted of point loads whose values were equal to the total abutment reactions divided by the number of superstructure bearing points. Different combinations of pile and superstructure bearing point configurations produced various maximum axial pile forces within a given pile group. The maximum axial pile force for the more practical configurations were compared to the pile forces when the gravity loads were assumed to be evenly distributed to all piles. The nominal axial pile factors, shown in Table 4.1, were developed to account for this difference between the two possible axial pile loads for various superstructure systems and pile layouts. The design axial pile force is equal to the total abutment reaction divided by the number of piles times the nominal axial pile factor in Table 4.1. Type 1 and Type 2 RRFC's refers to cars similar to those shown in Figures 1.3 and 1.4, respectively.

Table 4.1. Nominal axial pile factors for various superstructure systems.

Superstructure System	Nominal Axial Pile Factor
PCDT	1.40
BISB	1.35
RRFC (Type 1)	1.20
RRFC (Type 2)	1.40
Prestressed girder	1.30
Slab bridge	1.00
Quad-tee	1.50
Glulam girder	1.40

4.2.1.2. PILE BENDING MOMENT AND ANCHOR ROD FORCE

The lateral soil pressure distributions previously described are converted into distributed pile loads by multiplying the soil pressure by the pile spacing to obtain a force per unit length. It is assumed that the longitudinal braking force and transverse wind loads are transferred to the piles at the bearing location. The total longitudinal braking force per abutment is divided by the number of piles to obtain a concentrated force for each pile. Additionally, the transverse wind loads are also resolved into a concentrated pile force that is applied at the top of the pile to induce weak axis bending. The transverse wind on superstructure load per pile is calculated by multiplying the 50 psf wind pressure by half the span length, the superstructure elevation surface area, and then dividing by the number of piles. Similarly, the transverse wind on the bridge live load per pile is obtained by multiplying the 100 plf line load by half the span length and then dividing by the number of piles.

As previously noted in Chapter 3, two different lateral load analysis methods were compared. The linear method, presented by Broms [19, 20], produced comparable results to the non-linear computer analysis method. Since, the linear method can be easily used and incorporated into a foundation design template, it was selected for use in the design methodology for LVR bridge abutments. This allows the pile to be analyzed as a cantilever system. A lateral restraint system, consisting of a buried reinforced concrete anchor block, can be used to reduce the lateral loading effects. Also, a positive connection (i.e., the pinned end) between the superstructure and substructure uses the axial stiffness of the superstructure to transfer lateral loads among the substructures units.

The passive soil reactions for a single pile in both a cohesive and cohesionless soil resulting from external lateral loads are shown in Figures 3.7 and 3.8, respectively. The magnitude of this resistance depends on pile width parallel to the plane of bending and the properties of the soil. A uniform soil reaction is specified by Broms [19, 20] for cohesive soils, however no guidance on the exact shape of the soil reaction for cohesionless soils is provided. As a result, a parabolic shape was assumed. The total magnitude of the passive soil resistance equals the above ground lateral loadings.

If a lateral restraint system is not utilized, the maximum bending moment and deflection of the pile system is found using statics. The principal of superposition can be used to determine the combined effects of all the lateral pile loadings. The addition of a lateral restraint system creates a statically indeterminate system. Although there are several methods that can be used to solve this system, in this investigation an iterative, consistent deformation approach (in which the displacement of the lateral restraint system is equal to the displacement of the pile at the anchor location) was used. The two lateral restraint systems previously noted (a buried reinforced concrete anchor block and a positive bearing connection between the superstructure and substructure) were considered in this project.

To analyze each pile individually, the anchor rod axial stiffness per pile is calculated by equally distributing the total cross sectional area of all anchor rods for one abutment to each pile. However, an abutment wale as shown in Figures 3.1 and 3.2 must be provided so that the anchor rod forces can be transferred to the adjacent piles. An abutment wale is not needed if an anchor rod is connected to each pile. After the anchor rod axial stiffness per pile is established, the structural analysis of the system is performed using an iterative approach to determine the anchor rod force.

Once this force is known, the maximum bending moment and deflection along the length of the pile can be determined.

4.2.2. Internal Anchor Block Forces

The anchor block is analyzed as a continuous beam using simple supports that correspond to the location of the anchor rods. The net soil reaction imparted on the anchor block to resist the lateral substructure loads is represented by a uniform distributed load equal to the anchor rod force per pile multiplied by the number of piles and divided by the total length of the anchor block. The moment distribution method is used to determine the moment at the anchor rod locations, equilibrium equations are then used to determine the maximum internal shear and moment of the anchor block. Obviously, any structural analysis computer software could be used to determine the internal anchor block forces.

The support reactions obtained from the structural analysis will not necessarily be equal to the magnitude of the calculated anchor rod forces. Several factors can influence the distribution of the anchor rod forces within the anchor block. A higher percentage of the soil reaction load is distributed to the interior sections of the anchor block when compared to the cantilever anchor block length that extend beyond the end anchor rods (i.e., the end supports in the anchor block model). Additionally, if the anchor block extends well beyond the end anchor rod, the internal forces at the corresponding anchor rod location could control the design with higher internal moments and shears.

4.2.3. Miscellaneous Element Forces

The structural analysis of additional substructure elements such as the pile cap, abutment wale and backwall must also be performed. However, a design methodology for these additional elements is beyond the scope of this project.

The structural analysis of an abutment pile cap is similar to the process for the anchor block previously discussed. The pile cap is modeled as a continuous beam with simple supports that correspond to the locations of the piles. The total abutment reaction (including live load impact) is applied to the pile cap model as a series of concentrated forces that correspond to the superstructure bearing points. The magnitude of the concentrated forces are determined by either taking the total abutment reaction and dividing by the number of bearing points or using the tributary area above the superstructure bearing points. For a slab bridge, a uniform distributed load equal to the total abutment reaction divided by the length of the pile cap is used in place of the superstructure point loads. The moment distribution method is used as to determine the moments at the pile locations which are used to determine the maximum internal shears and moments in the pile cap.

Backwall components are typically composed of horizontal timber planks, vertically driven sheet piles, or some type of precast or cast-in-place concrete panels (Figure 3.4). The magnitude of the backwall loads are determined by computing the soil pressures acting at a point of interest and then applying these pressures to the tributary area of the backwall section. The abutment wale is analyzed as a continuous beam that spans between the supporting piles. There are two possible loading conditions for the abutment wale. If anchor rods are connected to the abutment wale, these rod forces are represented as point loads on the wale and act in the opposite direction of the backwall soil pressures. If the wale is located between the piles and backwall as shown in Figures 3.1 and 3.2, a uniformly distributed load that represents the total backwall load acting on a tributary area is applied to the abutment wale.

4.3. CAPACITY OF FOUNDATION ELEMENTS

The guidelines specified in AASHTO [29], the Iowa DOT BDM [10], the National Design Specification Manual for Wood Construction (NDS Manual) [30] and the American Institute of Steel Construction, Manual of Steel Construction (AISC Manual) [31] are all used to determine the capacities of the various foundation elements. This step in the design methodology is denoted as Part 'C' in Figure 4.1.

4.3.1. Pile Capacity

4.3.1.1. BEARING CAPACITY

In the approach used herein, piles are classified into three groups, end bearing, friction bearing, and combined friction and end bearing piles. End bearing piles develop the necessary vertical capacity from the bearing of the pile tip on a relatively hard foundation material. Estimated end bearing values (in psi) for various H-pile sizes and foundation materials as stated by the Iowa DOT FSIC are presented in Appendix B of Volume II. These values are correlated to the SPT blow count and include a factor of safety of 2.0. The pile capacity is equal to the product of the cross sectional pile area and the estimated end bearing value.

Friction piles develop the necessary resistance from the shear forces between the embedded pile surface and the surrounding soil. The magnitude of this bearing resistance varies significantly with pile type and soil type. The Iowa DOT FSIC also states estimated friction bearing values (in tons per foot) for various pile types and foundation materials. This information, which is correlated to the SPT blow count and includes a factor of safety of 2.0, is also included in Appendix B of Volume II. The values provided for timber piles are based on a pile diameter of 10 in. If a different pile diameter is used, an appropriate friction bearing value per foot can be obtained by dividing the values provided by 10 in. and multiplying by the actual pile diameter in inches. For friction piles, the bearing capacity is equal to the embedded pile length multiplied by the friction bearing value for the appropriate soil type.

The final pile bearing resistance category, friction and end bearing piles, combines the bearing components of the previous two bearing types. The total bearing value is equal to the sum of the end bearing and friction bearing resistances as previously described. If the end bearing material is bedrock, then there is a limited bearing capacity since the embedded pile length is a finite value.

4.3.1.2. STRUCTURAL CAPCITY

4.3.1.2.1. Steel Piles

The Iowa DOT BDM states that piles are to be designed using allowable stress design. During the investigation of the different substructure design methodologies used by the Iowa County Engineers, it was discovered that specifications provided in the AISC Manual were used by some engineers to investigate the structural capacity of steel piles subjected to both bending and axial loads. Therefore all equations used for the design methodology of steel piles in this section are taken from Sections 1.5 and 1.6 of the AISC Manual. It should be noted that similar design specifications summarized in this section are also provided in AASHTO. Two interaction equations are used to compare the ratios of the applied stress to allowable stress for combined axial and bending loads. Equation 4.2 is one of these two requirements for steel piles subjected to combined loads. In all equations for this section, the x-axis and y-axis refer to the pile bending axis that are parallel and perpendicular to the backwall face, respectively. It is also assumed that for steel piles, the x and y-axis refer to the strong and weak bending axis of the pile, respectively.

$$\frac{f_a}{F_a} + \frac{C_{mx}\, f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_b} + \frac{C_{my}\, f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_b} \le 1.0 \qquad (4.2)$$

where:

$C_{mx} = 0.6 - 0.4\dfrac{M_1}{M_2}$ = Strong axis buckling coefficient.

C_{my} = Weak axis buckling coefficient.

F_a = Allowable axial stress.

f_a = Applied axial stress.

F_b = Allowable bending stress.

f_{bx} = Applied strong axis bending stress.

f_{by} = Applied weak axis bending stress.

F'_{ex} = Strong axis Euler buckling stress divided by a factor of safety.

F'_{ey} = Weak axis Euler buckling stress divided by a factor of safety.

M_1 = Smaller of the two moments at the braced points.

M_2 = Larger of the two moment at the braced points.

The applied axial pile stress is equal to the axial pile load divided by the cross sectional area of the pile; the allowable axial pile stress is determined below. The applied strong and weak axis bending stresses are determined by dividing the maximum longitudinal and transverse pile moments by the strong and weak axis section modulus, respectively. The allowable bending stress is equal to 0.66 of the yield stress. The inverse of the two terms in parentheses in Equation 4.2 are the amplification factors that represent the secondary moments induced by the axial load and lateral deflection of the pile (P-Δ effect) [32]. The moment ratio used to define the strong axis buckling coefficient represents the strong axis bending moments at the two bracing points. This ratio is positive for reverse curvature. The passive soil reaction below the ground elevation that acts in the opposite direction of the above ground lateral pile loads induces reverse pile curvature. If the largest slenderness ratio (defined below for both the strong and weak axis) is less than the column buckling coefficient given by Equation 4.3, then Equation 4.4 is used to determine the allowable axial pile stress. If the largest slenderness ratio is greater than the column buckling coefficient, then Equation 4.5 is used with the appropriate slenderness ratio to determine the allowable axial pile stress.

$$C_C = \sqrt{\frac{2\pi^2 E}{F_y}} \tag{4.3}$$

where:

C_C = Column buckling coefficient.

E = Modulus of elasticity.

F_y = Pile yield stress.

$$F_a = \frac{\left[1 - \frac{(Kl/r)^2}{2C_C^{\,2}}\right]F_y}{\frac{5}{3} + \frac{3(Kl/r)}{8C_C} - \frac{(Kl/r)^3}{8C_C^{\,3}}} \tag{4.4}$$

where:

C_C = Column buckling coefficient (determined from Equation 4.3).

F_a = Allowable axial stress.

F_y = Pile yield stress.

K = Effective length factor (see Table 4.2).

Kl/r = Maximum slenderness ratio.

l = Length between braced points (see Table 4.2).

r = Radius of gyration.

$$F_a = \frac{12\pi^2 E}{23(Kl/r)^2} \tag{4.5}$$

where:

E = Modulus of elasticity.

F_a = Allowable axial stress.

K = Effective length factor (see Table 4.2).

Kl/r = Maximum slenderness ratio.

l = Length between braced points (see Table 4.2).

r = Radius of gyration.

The slenderness ratio used in Equations 4.4 and 4.5 is the maximum for either the strong or weak pile bending axis. A summary of the effective length factors and pile length between braced points to be used for the strong and weak axis both with and without a lateral restraint system is presented in Table 4.2.

Table 4.2. Effective length factors and pile lengths between braced points.

	No Lateral Restraint System Used		Lateral Restraint System Used	
	Strong Axis	Weak Axis	Strong Axis	Weak Axis
K	2.0	0.7	0.7	0.7
Distance between braced points	Distance from point of fixity to roadway	Distance from point of fixity to bearings	Distance from point of fixity to lateral restraint location	Distance from point of fixity to bearings

The strong and weak axis Euler buckling stresses used in Equation 4.2 are found by using the strong and weak axis slenderness ratios, respectively in Equation 4.5. Equation 4.6 lists the second requirement for steel piles subjected to both axial and bending loads.

$$\frac{f_a}{0.60\,F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \le 1.0 \tag{4.6}$$

where:

f_a = Applied axial stress.

F_{bx} = Allowable strong axis bending stress.

f_{bx} = Applied strong axis bending stress.

F_{by} = Allowable strong axis bending stress.

f_{by} = Applied weak axis bending stress.

F_y = Pile yield stress.

4.3.1.2.2. Timber Piles

Guidelines specified by AASHTO and the NDS Manual were used to develop the design methodology for timber piles. The material strengths of timber vary significantly with the type of species, member size, member shape, loading conditions and surrounding environmental conditions. Therefore, timber modification factors are used to account for these variables. All equations and modification factors used in this section are described greater detail in AAHSTO, Section 13. Both AAHSTO and the NDS Manual state that when necessary, round timber members can be treated as square members with an equivalent cross sectional area. Additionally, the diameter used to calculate the modification factors and the allowable stresses should be based on a representative cross sectional area of the pile. Since timber piles are tapered with the tip being smaller than the butt, a representative pile diameter is calculated using Equation 4.7 to account for the varying cross section of the pile. Section 4165 of the Iowa DOT Standard Specifications [33] provides a table of minimum butt and tip diameters for timber piles.

$$d_{rep} = d_{min} + 0.33\left(d_{max} - d_{min}\right) \tag{4.7}$$

where:

d_{max} = Maximum pile diameter (i.e., the pile butt).

d_{min} = Minimum pile diameter (i.e., the pile tip).

d_{rep} = Representative pile diameter.

AASHTO refers to Chapter 3 of the NDS Manual for the design of timber piles subjected to both axial and bending loads. Equation 4.8 (from Section 3.9 of the NDS manual) is used for timber piles subjected to both bending and compressive loads. In Equation 4.8, the x-axis and y-axis refer to the pile bending axis that is parallel and perpendicular to the backwall face, respectively.

$$\left(\frac{f_C}{F'_C}\right)^2 + \frac{f_{bx}}{\left(1-\frac{f_C}{F'_{ex}}\right)F'_{bx}} + \frac{f_{by}}{\left(1-\frac{f_C}{F'_{ey}}-\left(\frac{f_{bx}}{F_{bE}}\right)^2\right)F'_{by}} \leq 1.0 \tag{4.8}$$

where:

F_{bE} = Bending buckling stress.

F'_{bx} = Allowable x-axis bending stress.

f_{bx} = Applied x-axis bending stress.

F'_{by} = Allowable y-axis bending stress.

f_{by} = Applied y-axis bending stress.

F'_C = Allowable compressive axial stress.

f_C = Applied compressive axial stress.

F'_{ex} = X-axis buckling stress.

F'_{ey} = Y-axis buckling stress.

The applied axial pile stress is equal to the axial pile load divided by the representative cross sectional area of the pile and the applied x-axis and y-axis bending stresses are equal to the respective maximum pile moments divided by the section modulus. Since timber piles have a circular cross section, there is no difference between the x-axis and y-axis section properties. The allowable compressive axial stress is determined using Equation 4.9. This equation involves a tabulated axial compressive stress and a series of multiplication factors. The tabulated axial compressive stresses provided by AAHSTO depend on the type of timber species and the structural grade. Section 4165 of the Iowa DOT Standard Specifications states that all timber piles shall be structural grade lumber of either southern pine or douglas fir species. All multiplication factors discussed herein are applicable to these conditions.

$$F'_C = F_C\, C_M\, C_D\, C_P \tag{4.9}$$

where:

C_D = Load duration factor.

C_M = Wet service factor.

C_P = Controlling column stability factor.

F'_C = Allowable compressive stress parallel to the grain.

F_C = Tabulated compressive stress parallel to the grain.

The wet service factors are classified by member size and species. For timber piles, a five inch square member or larger is used to obtain wet service factors of 1.0 and 0.91 for southern pine and douglas fir species, respectively. For this project, all load applications are considered to be permanent, thus a load duration factor of 0.90 is used. As shown in Equations 4.10 and 4.11, the column stability factor depends on the effective pile length previously described and shown in Table 4.2. The x-axis and y-axis correspond to the strong and weak axis values, respectively in Table 4.2. The effective column length that yields the smaller column stability factor should be used in Equation 4.9.

$$C_P = \frac{1 - F'_e/F_C^*}{2c} - \sqrt{\frac{\left(1 + F'_e/F_C^*\right)^2}{(2c)^2} - \frac{F'_e/F_C^*}{c}} \tag{4.10}$$

where:

- c = Member type adjustment factor.
- C_P = Column stability factor.
- F_c^* = Allowable compressive stress computed using Equation 4.9 without column stability factor.

$$F'_e = \frac{K_{cE}\,E'}{\left(l_e/d\right)^2} \tag{4.11}$$

where:

- d = Equivalent square dimension.
- E' = Tabulated modulus of elasticity multiplied by the wet service factor.
- F'_e = Buckling stress.
- K_{cE} = Timber grading factor.
- l_e = Effective column length.

For visually graded, round timber piles, values of 0.85 and 0.30 are used for the member type adjustment factor and timber grading factor, respectively. The allowable bending stress is calculated using Equation 4.12. For this design methodology, the allowable bending stress for the x-axis and y-axes are equal in value.

$$F'_b = F_b\,C_M\,C_D\,C_L\,C_f \tag{4.12}$$

where:

- C_D = Load duration factor.
- C_f = Form factor.

C_L = Beam stability factor.

C_M = Wet service factor.

F'_b = Allowable bending stress.

F_b = Tabulated bending stress.

AASHTO provides a list of tabulated unit bending stresses for various timber species and lumber grades. A wet service factor of 1.0 is used for all timber piles that have an equivalent cross sectional area greater than or equal to a five-inch square member. As before, all load applications are considered to be permanent, thus a load duration factor of 0.90 is used. For timber members with a round cross section, a form factor equal to 1.18 is used. Finally, for members whose width does not exceed its depth, the beam stability factor is equal to 1.0.

Equations 4.13 and 4.14 are both used in the y-axis, secondary moment amplification (P-Δ) factor of Equation 4.8.

$$F_{bE} = \frac{K_{bE}\, E'}{{R_B}^2} \tag{4.13}$$

where:

E' = Tabulated modulus of elasticity multiplied by the bending wet service factor.

F_{bE} = Bending buckling stress.

K_{bE} = Timber grading factor.

R_B = Bending slenderness ratio.

$$R_B = \sqrt{\frac{l_e\, d}{b^2}} \tag{4.14}$$

where:

b = Member width.

d = Member depth.

l_e = Effective pile length.

R_B = Bending slenderness ratio.

The NDS manual cites a timber grading factor value of 0.439 for visually graded lumber. Since the bending slenderness ratio is used in Equation 4.8 to compare the x-axis applied and buckling bending stresses, the effective pile length for Equation 4.14 should also correspond to the x-axis direction. For round timber piles the pile depth and width are equal to the equivalent square dimension previously discussed.

4.3.2. Anchor Block Capacity

In addition to the design of the piles, the capacity of the anchor block system must also be verified. This includes the determination of the anchor block structural capacity and the passive resistance of the surrounding soil. Variables such as the anchor rod force per pile, the elevation of the anchor system, anchor rod properties, and backwall width that were previously discussed are also used to determine the capacity of the anchor block.

4.3.2.1. LATERAL CAPACITY

The capacity of the soil surrounding the anchor block must be verified to ensure that it is capable of providing the necessary lateral resistance. The maximum efficiency of the anchor system is achieved when the anchor block is positioned beyond the passive and active soil zones as shown in Figure 4.6 [12].

The anchor block system develops its lateral capacity from the mobilized soil pressures that acts on the vertical anchor block face as shown in Figure 4.7. The soil pressure distributions are a function of the surrounding soil properties and the depth of the anchor block with respect to the roadway surface. The magnitude of the maximum passive and active soil pressures acting on the anchor block face is based on the Rankine earth pressure theory which assumes that no shear forces exist between the vertical anchor block face and surrounding soil [12]. It should be noted that

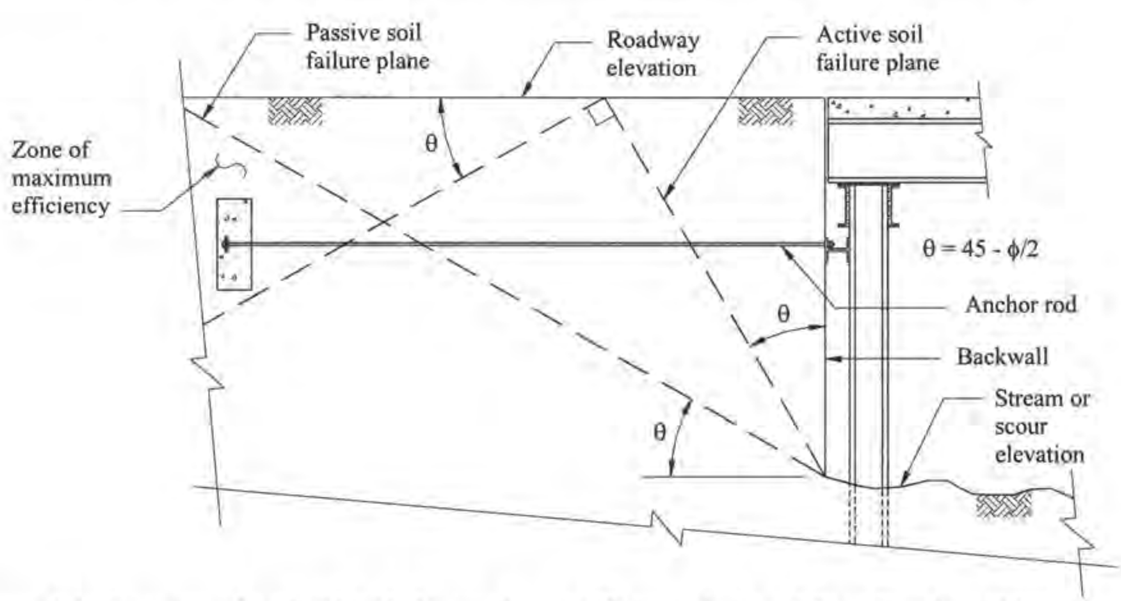


Figure 4.6. Location of anchor block for maximum efficiency [adapted from Bowles, 1996].

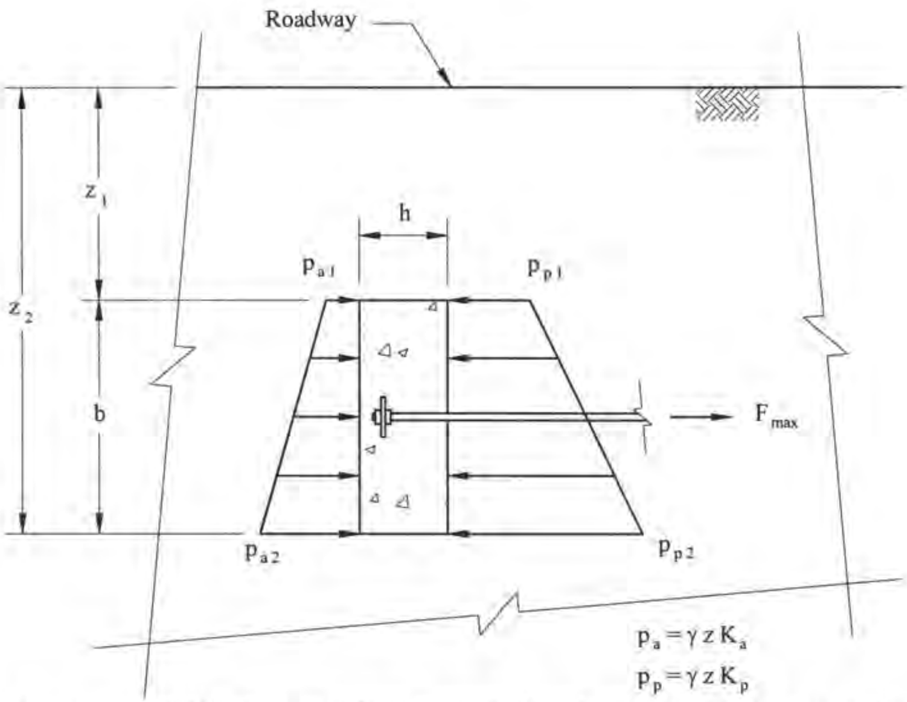


Figure 4.7. Soil pressure distribution used to determine the lateral anchor block capacity [adapted from Bowles, 1996].

increasing the depth of the anchor block below the roadway will increase the lateral capacity, however this will reduce the anchor systems effectiveness in reducing the maximum pile moment. If an inclined anchor rod is used, the Coulomb theory, which accounts for shear forces on the anchor block face, should be utilized to determine the lateral capacity of the soil surrounding the anchor block.

The magnitude of the maximum lateral capacity is calculated using Equation 4.15. Bowles [12] recommends a factor of safety of 1.5 when calculating the soil resistance (not included in Equation 4.15). It should be pointed out that one must ensure that the backfill soil is carefully compacted around the anchor block so that the passive and active pressures can be fully mobilized [12]. The total lateral capacity of the anchor block system per pile is equal to the anchor resistance per foot (i.e., Equation 4.15) multiplied by the pile spacing.

$$F_{max} = \frac{\gamma b}{2}(z_1 + z_2)(K_P - K_a) \tag{4.15}$$

where:

- b = Anchor block height.
- F_{max} = Maximum lateral anchor block capacity (force per unit length).

$K_a \quad = \dfrac{1-\sin\phi}{1+\sin\phi}$ = Rankine active earth pressure coefficient.

$K_p \quad = \dfrac{1+\sin\phi}{1-\sin\phi}$ = Rankine passive earth pressure coefficient.

z_1 = Distance from roadway grade to the top of anchor block.

z_2 = Distance from roadway grade to the bottom of anchor block.

ϕ = Soil friction angle.

γ = Soil unit weight.

4.3.2.2. STRUCTURAL CAPCITY

Once the lateral capacity of the anchor block has been verified, the structural capacity must be investigated. The anchor block is designed using reinforced concrete design practice described in AASHTO, Section 8. This includes designing the flexural and shear reinforcement in addition to checking the development length requirements for the flexural reinforcement, the ductility, and the minimum reinforcement requirements. It should be noted that the internal anchor block bending loads induced by the anchor rods and soil pressure distributions act on a plane that is parallel to the backwall face. Therefore the effective depth of the concrete used for flexure design is determined using the dimension of the horizontal anchor block face dimension, 'h'.

4.3.3. Miscellaneous Substructure Elements

The capacity of additional substructure elements such as the pile cap, abutment wale, and backwall must also be determined. As previously mentioned, a design methodology for these additional elements was beyond the scope of this project.

A reinforced concrete pile cap is designed using AASHTO, Section 8 whereas Section 10 is used for the design of a steel pile cap. The structural capacity of the abutment wale should also be determined using AASHTO Section 10 and Section 13 for steel and timber wales, respectively.

4.4. DESIGN CHECKS OF FOUNDATION ELEMENTS

Once the internal element loads and capacities have been determined, the adequacy of the substructure system must be checked. In general, this consists of verifying that the system capacity is greater than the loads. This step in the design methodology for LVR bridge abutments is denoted as Part 'D' in Figure 4.1. This section provides specific design requirements for the pile and anchor system.

The structural capacity of both steel and timber piles is not computed directly using the design methodology presented in this report. Interaction requirements are used to compare the ratios of applied to allowable stresses for combined bending and axial loadings. For steel piles, if Equation 4.2 or 4.6 yield a value less than 1.0, the pile is considered structurally adequate. This requirement is the same for timber piles, however Equation 4.8 is used. If the interaction equation requirement is not satisfied, an alternative substructure configuration must be used.

The pile bearing capacity must also be larger than the axial pile load. However, additional bearing requirements are cited by AASHTO and the Iowa DOT BDM. Both sources state that the maximum applied axial steel pile stress must not exceed 25 percent the yield stress. Section 6.2.6 of the Iowa DOT BDM provides more detailed axial pile stress requirements for both steel and timber piles based on the type of bearing resistance and the type of foundation material. The maximum allowable axial pile stress for a friction bearing steel pile is equal to 6 ksi. For end bearing steel piles, the maximum allowable axial pile stress is equal to 6 and 9 ksi for end bearing foundation material with a SPT blow count less than and greater than 200, respectively. Finally, the maximum axial pile stress for combined friction and end bearing steel piles is equal to 9 ksi for an end bearing foundation material with a SPT blow count between 100 and 200. The maximum allowable axial pile stress is equal to 6 ksi for all other combinations of friction and end bearing foundation materials. For timber piles, the Iowa DOT BDM states that the applied axial pile load must be less than 20 tons for pile lengths between 20 and 30 ft and 25 tons for pile lengths between 35 and 55 ft.

The capacity of the anchor system must also be verified. The applied anchor rod stress must be less than the allowable anchor rod stress defined in the AISC Manual as 60 percent of the yield stress. The maximum passive resistance of the soil surrounding the anchor block (per foot of length) is obtained from Equation 4.15. This capacity per foot is multiplied by the pile spacing and must be greater than the required anchor force per pile previously discussed. It is recommended that the total length of the anchor block be greater than or equal to the number of piles multiplied by the pile spacing. In order to satisfy the structural design requirements, the internal anchor block shear and bending loads must be less the structural capacity of the anchor block determined using AASHTO reinforced concrete guidelines.

5. ALTERNATIVE LOW-VOLUME ABUTMENT SYSTEMS

The literature search revealed several alternative abutment systems that may be of interest to Iowa Engineers. These systems are well established in a particular geographic region or for a specific use, however none of them have been used as a bridge abutment system in Iowa. The alternative abutment systems include micropiles, geosynthetic reinforced soil (GRS) structures, Geopier foundations, and sheet pile abutments. Since these are economical and provide advantages over the traditional deep foundations systems currently used (i.e., driven piles), they show promise for numerous sites in Iowa. As noted in Chapter 3 and Chapter 7, it is proposed that several of these systems be tested in demonstration projects.

5.1. MICROPILES

Micropiles originated in Italy in the early 1950's and are used to strengthen and stabilize existing structure foundations. The term "micropile" is one of many terms used to describe a small diameter bored injection pile. Other terms include: minipile, root pile, pinpile, drilled-in-pier pile and drilled cast-in-place concrete pile [34, 35]. The term micropile will be used herein.

A micropile is typically defined as a small diameter structural element that was constructed by boring a hole in the soil and filling it with steel reinforcement and either gravity flow or pressurized grout. The steel reinforcement typically consists of either steel reinforcement bars or a tubular casing. Micropile lengths of almost 100 ft with diameters ranging from 3.9 to 11.8 in. have been documented [34]. Depending on the soil conditions and pile size, a micropile can have a bearing capacity up to 225 kips. This relatively large capacity is developed from the fictional forces between the grout and the surrounding soil [36].

Significant micropile usage began in the United States in the late 1970's [35]. California is one of the leading states in the use of micropile foundations. Many existing foundations in the earthquake prone region require retrofitting to meet new seismic design code requirements. Micropiles have both substantial tensile and compressive capacities making them ideal for these situations. Micropiles can be easily incorporated in an existing structure by either drilling holes in the existing foundation or tying a new pile cap into the existing structure [37].

As previously discussed, micropiles were originally developed to underpin or strengthen existing structure foundations in urban areas where excavation or driven piles were not feasible alternatives. Driven piles require more space and overhead clearance when compared to the minimum 8 ft clearance required for the installation of some micropiles. Also, the excessive vibrations associated with driving piles can influence the surrounding soil and initiate additional settlement [35].

There are many situations when a micropile system could be more cost effective than driven piles. The equipment used for micropile installation is relatively small compared to pile driving equipment and is therefore more mobile. Micropiles are also ideally suited for fragile environmental areas since the installation equipment produces a relatively small amount of noise and vibrations. Another advantage of micropiles is that they can be installed in situations in which traditional driven piles may not be practical. This includes the presence of compressible and expansive soil layers. Downdrag and uplift forces are not as influential on micropile foundations due to the relatively small surface area of the piles. Additionally, the presence of cobbles, boulders and other subsurface obstructions are not as troublesome for micropiles as they may be for driven piles [37].

Micropiles work well for many situations however there are some restrictions. The small size of micropiles limit their lateral load and bending capacities. Alternatives include the use of a battered micropile to resist lateral loadings or replacing the single bar reinforcement with structural steel tubing. Another restriction of micropiles is the special techniques required for installation, this includes various drilling techniques, reinforcement types, grout mixtures, and grout placement procedures. If a micropile is not properly designed for the site conditions, or if the contractor does not have sufficient experience with installing micropiles, the structural and bearing capacity of the micropile can be compromised [37].

Micropile installation procedures may require an experienced contractor, however the techniques and equipment required are generally no different than what is required for the installation of ground anchors, soil nails and grout holes. The general construction sequence for micropiles using a drill casing is shown in Figure 5.1. First, a hole is drilled to the appropriate depth. A drill casing is inserted into the hole to maintain the shape as the depth increases. Once the desired depth has been obtained, the drill bit is removed and the casing is left in place. Next, gravity flow grout is placed in the hole in addition to any steel reinforcement. Finally, an additional amount of grout is placed under pressure as the drill casing is raised to the final height. The finished micropile is then tied into a new or existing structure foundation. The increased grout pressure will create a grout bulb with an increased surface area. Additionally, the lateral pressure increases the bearing capacity of the pile by improving the ground-to-ground bond. The drill casing can be left in place to act the pile reinforcement [37].

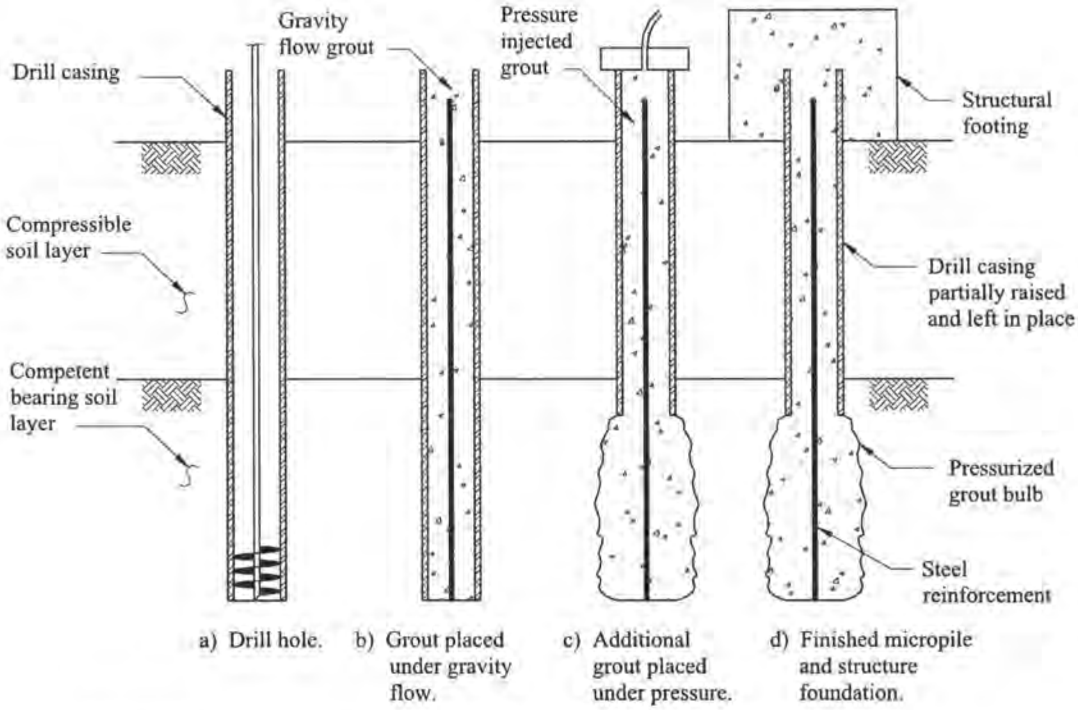


Figure 5.1. Micropile construction sequence [adapted from FHWA, 2002].

The design of a micropile includes several aspects of structural and geotechnical engineering. The bearing capacity of the pile, including friction and end bearing must be investigated. The FWHA publication; *Micropile Design and Construction Guideline* [37] includes a table of grout-to-ground bond design strengths for different soil and rock types. The structural design of micropiles typically controls for most situations since the cross sectional area of the micropile is relatively small. Also, the connection of the micropile to the structure must be investigated to ensure that the loads can be safely transferred to the micropile foundation [37].

The FHWA provides micropile foundation design examples for both service load design (SLD) and load factor design (LFD) approaches in accordance with AASHTO. These examples include length and embedment calculations, bearing and structural capacity design checks, buckling and lateral load considerations, load factors, strength reduction factors, and serviceability limits. Currently, most geotechnical engineers use the SLD method, however engineers are changing to the LFD method and the new load resistance factor design (LFRD) [37].

5.2. GEOSYNTHETIC REINFORCED SOIL BRIDGE ABUTMENTS

A geosynthetic reinforced soil (GRS) bridge abutment is a retaining wall with layers of geosynthetic material attached to the front wall face that extends back between lifts of well-compacted backfill as shown in Figure 5.2. Typically, a shallow bridge spread footing rests directly

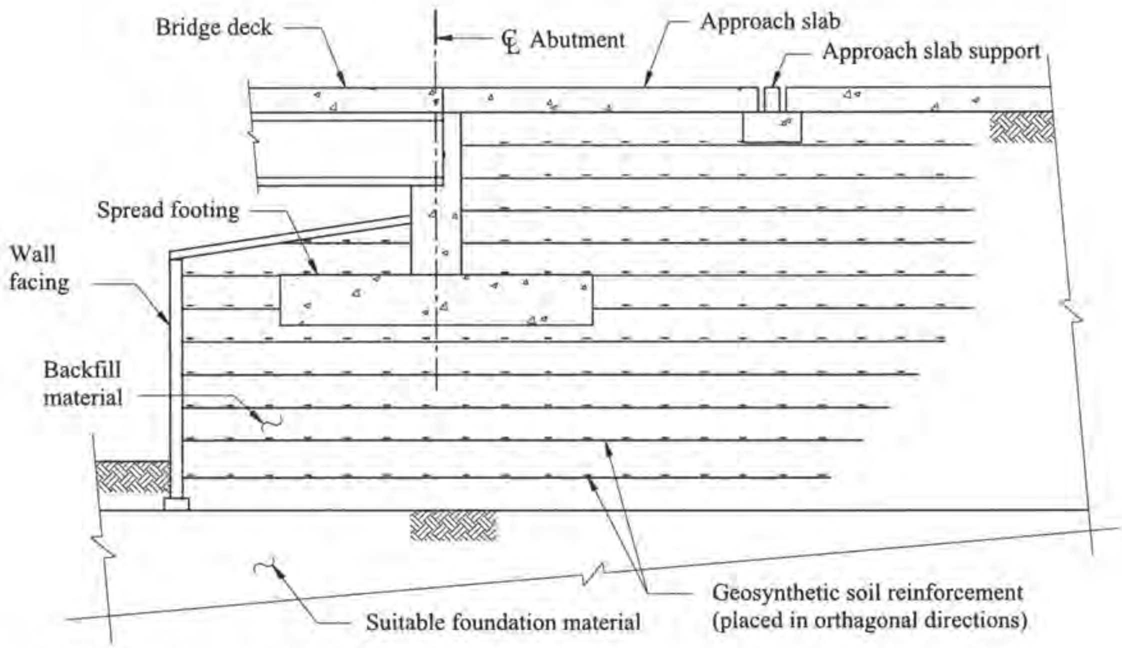


Figure 5.2. Cross-section of a GRS bridge abutment [adapted from Abu-Hejleh et al. 2000].

on the GRS mass several feet behind the wall face [38]. The wall facing consists of either rigid cast-in-place concrete, or a flexible material such as modular concrete blocks, timber planks, or gabions. Typical geosynthetic reinforcement consists of polymeric geosynthetic geotextiles or geogrids that are placed in orthogonal directions [39]. The foundation material below various GRS structures can range from bedrock to a fairly soft soil [40].

Various case histories have demonstrated the many advantages associated with a GRS bridge abutment. Some issues, such as aesthetics, do not significantly influence the design or cost. GRS bridge abutment wall facing blocks can be designed to present a structure that is more pleasing to the public when compared to a cast-in-place reinforced concrete bridge abutment [38]. Most of the major advantages associated with a GRS bridge abutment relate to the potential cost savings. A GRS bridge abutment can be constructed in a relatively short time using light construction equipment and simple techniques. Heavy equipment such as cranes, drilling equipment, and pile driving machinery are not required. The only equipment required is a dump truck, front-end loader, compaction equipment, and a backhoe for excavation. The use of local labor and small equipment combined with a quick construction time generally results in a significant savings in construction costs [41].

Additional cost savings for a GRS bridge abutment can be realized by the reduction in differential settlement between the bridge and approaching roadway thus eliminating "the bump at the end of the bridge". Different settlement rates between the bridge foundation system (deep or shallow) and the approach roadway fill are typically the cause for this differential settlement. Past attempts to solve this problem have included extension of the wingwalls to further contain the backfill soil, using a stronger/stiffer approach slab, and using granular backfill soil to limit the magnitude of the settlement [38]. As shown in Figure 5.2, the geosynthetic reinforcement extends well beyond bridge abutment spread footing, thus the bridge foundation and approaching roadway are both supported by the same system. The additional approach slab support as shown in Figure 5.2 is to help reduce differential settlement in a GRS bridge abutment.

There are certain situations in which a GRS bridge abutment may not be a feasible alternative. For example, the front face of the GRS bridge abutment wall does not extend very far below the ground line. Therefore, the GRS mass should be placed on a coarse, non-scour susceptible material or should only be used in situations where the potential for scour does not exist [42]. Additionally, GRS bridge abutments can tolerate differential settlement thus exhibiting good seismic performance [38]. However, if the total settlement is expected to greater than three inches, deep foundation elements should be considered [42].

The Founders/Meadow bridge abutment in Colorado was the first GRS bridge abutment constructed in the United States for large volumes of traffic; this bridge was opened in 1999 [44]. Other GRS bridge abutments have been built by several different government agencies such as the FHWA, the Colorado DOT, as well as the California and Alaska DOT's in conjunction with the United States Forest Service [40, 41, 44]. These GRS bridge abutments were for either small forest park roads, trail bridges, or for experimental purposes.

As previously described, the application of a GRS structure as a bridge foundation is a relatively new idea. Therefore standard design guidelines have not been created. Based on the performance of the experimental and in service GRS bridge abutments, some recommendations can be made. One important factor associated with a GRS bridge abutment is the condition of the backfill soil. The backfill material should consist of a course-grained soil with a high soil friction angle that is compacted with a 95 percent compactive effort [41, 43]. The Colorado DOT also recommends the construction of the backfill should take place in the drier, warmer months instead of the cold winter season. It may be possible that excess moisture could become trapped and freeze in the backfill soil during the winter months. When the temperature increases, thawing could create an outward wall displacement [42].

The geosynthetic reinforcement is a key element in a GRS mass that promotes a significant increase in vertical bearing capacity. It has been documented that the strength of the geosynthetic reinforcement is not as influential as the vertical spacing of the reinforcement. A smaller vertical spacing allows for more shear interaction between adjacent layers of reinforcement. This smaller spacing also requires smaller soil backfill lifts allowing better compaction control [42]. It has also been stated that a smaller vertical spacing will increase the overall stiffness of the reinforced soil mass thus reducing the associated creep deformations [45]. One final design recommendation states an allowable footing bearing pressure of 3.1 ksf (converted from 150 kPa) for GRS bridge abutments similar to the Founders/Meadows site. Additionally, the allowable footing bearing pressure could be increased to 4.2 ksf (converted from 200 kPa) if a smaller vertical reinforcement spacing is utilized [41, 43].

Recently, researchers in Japan began using preloaded and prestressed GRS bridge supports. As shown in Figure 5.3, a preloaded and prestressed GRS structure is constructed with rigid reaction blocks placed on the top and bottom of the GRS mass that are connected with vertical tie rods. A hydraulic jacking system is used to apply tension to the tie rods thus inducing a prestress in the GRS mass. A series of cyclic loadings are typically applied up to the final prestressing load. The cyclic loading, as well as the final prestressing load, increases the overall stiffness of the GRS mass thus creating a nearly elastic structure for normal service conditions. Preloading and prestressing can also help limit creep deformations from sustained vertical loads, the residual compression from cyclic service loading, and the magnitude of the vertical deflection from live loads [46].

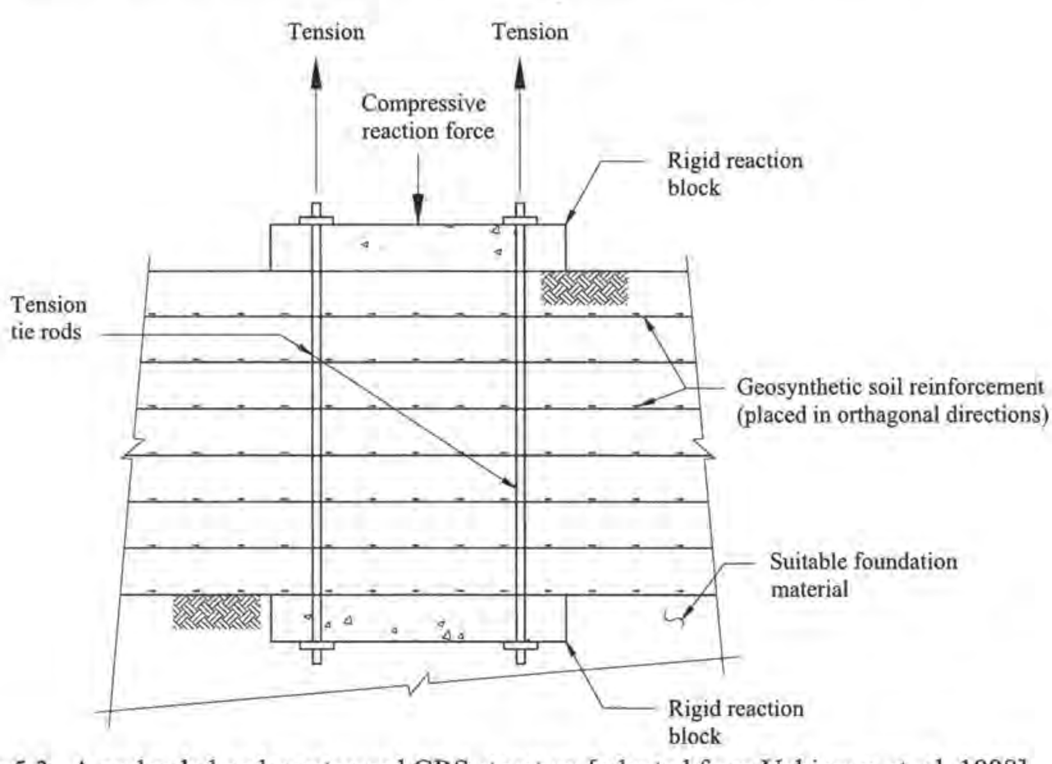


Figure 5.3. A preloaded and prestressed GRS structure [adapted from Uchimura et. al, 1998].

The first preloaded and prestressed GRS bridge pier was put into service in 1997. For this initial project, a GRS bridge pier and abutment were constructed. The pier was preloaded and prestressed, however the abutment was not. This allowed for a direct comparison of the two GRS systems. The construction of the GRS bridge pier took five workers a total of five days to complete and the duration of the preloading process lasted a total of 72 hours. Structural monitoring under service loads has revealed that the preloaded and prestressed GRS bridge pier has behaved nearly elastically whereas the GRS bridge abutment has shown a relatively larger residual compression. The Japanese researchers believe that the GRS bridge abutment will require premature maintenance work whereas the preloaded and prestressed GRS bridge pier will not [46].

5.3. GEOPIER FOUNDATIONS

Geopier foundations, or rammed aggregate piers, are a type of specially compacted aggregate columns that can be used to vertically reinforce a soil profile thus allowing a shallow spread footing foundation to be used in poor soil conditions. Geopier foundations are being used to control foundation settlement, provide uplift capacity, and to stabilize soil slopes. Geopier foundations are constructed using a unique technique that imparts lateral stress on the surrounding soil which increases the vertical bearing capacity and reduces the magnitude of total settlement [47]. Geopier elements are designed to improve the surrounding soil conditions; they do not support the foundation loads as independent structural members, therefore they do not need to extend to deeper, more competent soil layers [48]. These advantages allow Geopier foundations to be an effective and cost-competitive alternative. In certain situations, Geopier foundations have been shown to provide a 40 to 60 percent cost savings when compared to deep foundations [47]. For example, a Geopier foundation with lengths ranging from 7 to 9 ft was used in the construction of a parking garage in place of 75 ft driven piles at a cost savings of over 50 percent [49].

One of the biggest advantages associated with a Geopier foundation is that it can be used in poor soil conditions where settlement maybe a concern. In these situations, typical foundation solutions could include the excavation and replacement of existing weak soil layers or driving piles to bedrock. Typical Geopier foundations are less than 20 ft in length and have been documented to work in a variety of situations including soft organic clays, peat, loose silt, uncompacted fill soils, debris fill soils, stiff to very stiff clays, and medium dense to dense sands. Geopier foundations can increase the bearing capacity of weak soils to a level at which the construction of a structure is feasible [49].

The Geopier foundation construction sequence is shown in Figure 5.4. The first step in the construction sequence involves drilling a hole with typical dimensions ranging from 24 to 36 in. in diameter and 6 to 23 ft deep. A layer of crushed, clean aggregate is placed in the bottom of the hole and then compacted using a high-energy, low frequency tamper. This causes the formation of an aggregate bulb at the base of the shaft that effectively increases the length of the Geopier element by about one shaft diameter. Finally, the shaft void is filled with 12 in. thick lifts of well-graded aggregate. A picture of the equipment typically used for the compaction process is shown in Figure 5.5 [50, 51].

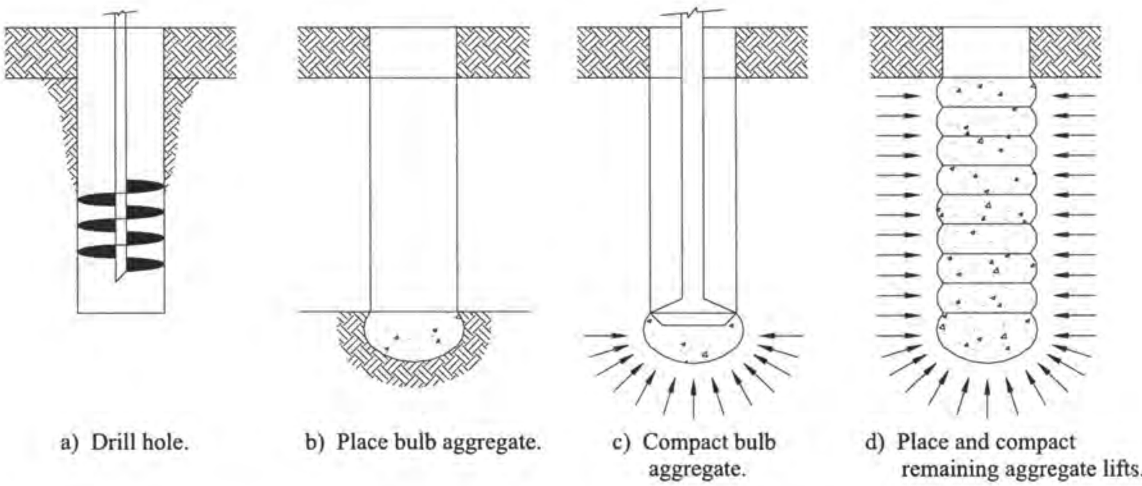


Figure 5.4. Geopier element construction sequence [adapted from Wissman et. al, 2000].

Figure 5.5. High-energy, low-frequency hammer used for the construction of Geopier foundations.

Geopier foundations improve the in-situ soil conditions by increasing the vertical and horizontal effective stress in the surrounding soil. The beveled shape of the high-energy hammer forces the compacted aggregate lifts vertically and laterally against the shaft walls. The increase in lateral soil stress corresponds to an increase in the soil stiffness. Thus, the soil profile behaves in a more elastic manor reducing both the immediate and long-term settlement. The vertical compactive effort also creates a stiffened column element that increases the overall average soil friction angle which correlates to an increase in bearing capacity [49, 50, 51].

Typically, Geopier foundations occupy about 30 to 40 percent of the foundation plan area and can increase the allowable soil bearing capacity between 5 and 9 ksf [51]. Geopier foundations can also be designed to provide an uplift capacity of up to 48 kips per element. For this situation, there is a direct connection between the Geopier element and the structure foundation as illustrated in Figure 5.6. Vertical tie rods are connected to a steel plate near the bottom of the Geopier element. The relatively high shear stress that develops between the aggregate and the shaft wall from the compactive effort allows the Geopier element to behave as a high capacity friction pile. Tensile uplift tests have documented that a Geopier foundation behaves essentially elastically in silty sands. Tensile uplift tests conducted in clayey soils revealed plastic deformations of less than one inch [47].

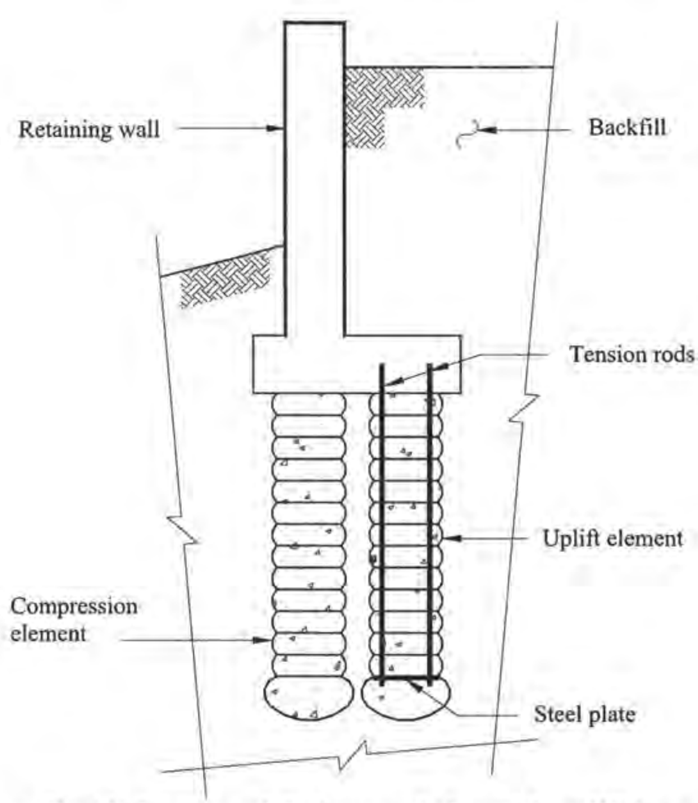


Figure 5.6. Retaining wall with Geopier uplift elements [adapted from White et. al, 2001].

5.4. SHEET PILE ABUTMENTS

The use of sheet piles in the United States has traditionally been limited to retaining wall type structures. However, sheet piles have been used in Europe as the main foundation elements in road bridge abutments for over 50 years [52]. In the past decade, this system has been used in various states in the United States. Sheet piles not only have the capacity to resist the moment from lateral soil pressures, but also vertical gravity loads [53], which has many advantages. For typical LVR bridge abutments in Iowa, sheet piles are placed behind the foundation piles to act as a retaining wall. The use of a bearing sheet pile eliminates the need for separate backwall and the foundation piles. Details commonly associated with a sheet pile abutment are shown in Figure 5.7.

Another advantage of sheet piles abutments is reduced amount of construction. Sheet piles do not require a significant amount of earthwork at the bridge site. For example, an earth embankment on the streamside face of the abutment is not required. The reduction in earthwork also permits one to use fewer construction stages in heavily congested areas in addition to allowing continual traffic flow during the bridge substructure construction. Sheet pile abutments can also be

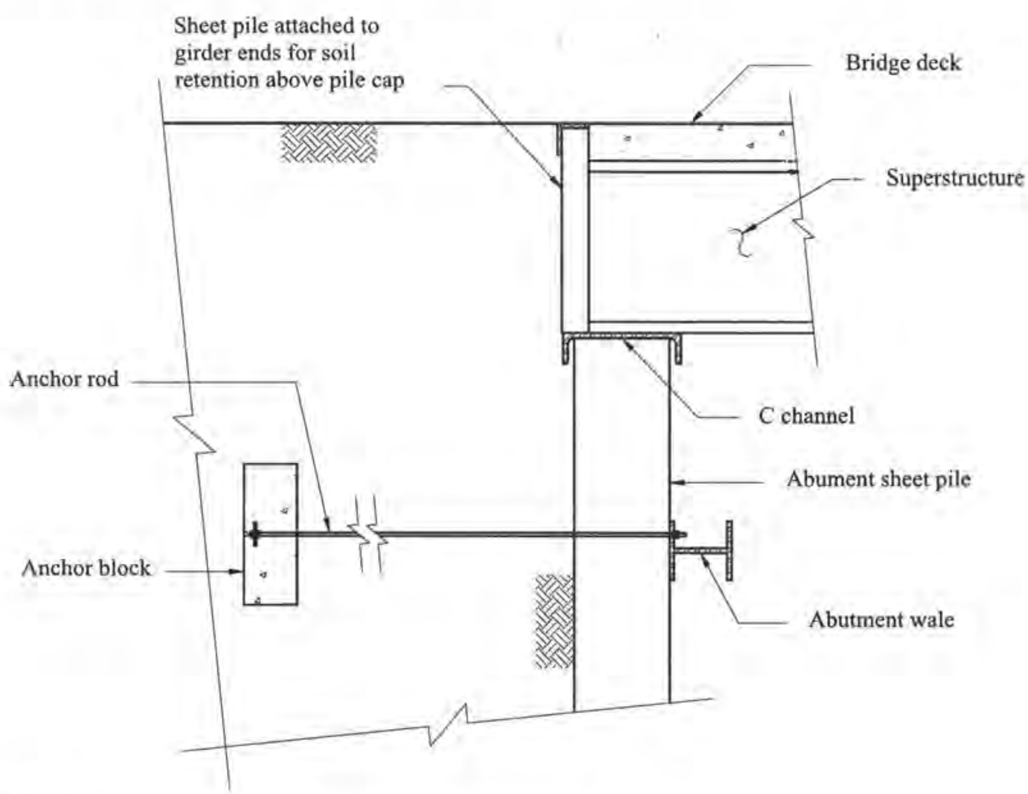


Figure 5.7. Cross-section view of a sheet pile abutment.

more cost-effective for LVR road bridges in situations where materials might not be readily available. A county could stockpile sheet pile sections instead of having to pay additional costs associated with associated with a sheet pile abutment. For example, the sheet piles provide sufficient scour protection without any additional protective measures or regular maintenance requirements. The low-maintenance advantage in addition to the relatively simple construction method make it possible for county forces to install and maintain the abutments without any external assistance [53].

In order to accurately estimate the horizontal sheet pile loads in addition to the lateral and bearing capacity, a detailed subsurface investigation including borings and soil tests should be performed. Once the foundation loads have been determined, the structural adequacy of a sheet pile section can be established. If a single row of sheet piles is not sufficient for the substructure, there are several alternatives. Box sheet piles, which are two u-shaped sheet piles placed back-to-back, can be used to create a series of pipe piles that are connected to the adjacent sheet piles to form the soil retaining structure as shown in Figure 5.8 [53]. These box piles will increase the cross-sectional area of the wall in addition to increasing the flexural capacity of the system. Also, a lateral restraint system can be used to reduce the lateral loads effects as previously discussed in Chapter 4.

In addition to the structural capacity of the sheet pile abutment, the lateral and bearing capacity of the soil must also be verified. Bustamante and Gianeselli [54] provide basic design equations to determine both the end bearing and skin friction resistance for sheet piles in dense sands and plastic clays based on results from experimental tests. These design equations have been correlated to SPT, pressuremeter, and cone penetration test results.

Sheet pile abutments can be used in a variety of situations. In addition to stub abutments, sheet piles can also be used for integral abutments, which call for the use of a flexible foundation element. Sheet piles can accommodate the longitudinal thermal movements and the end rotation of

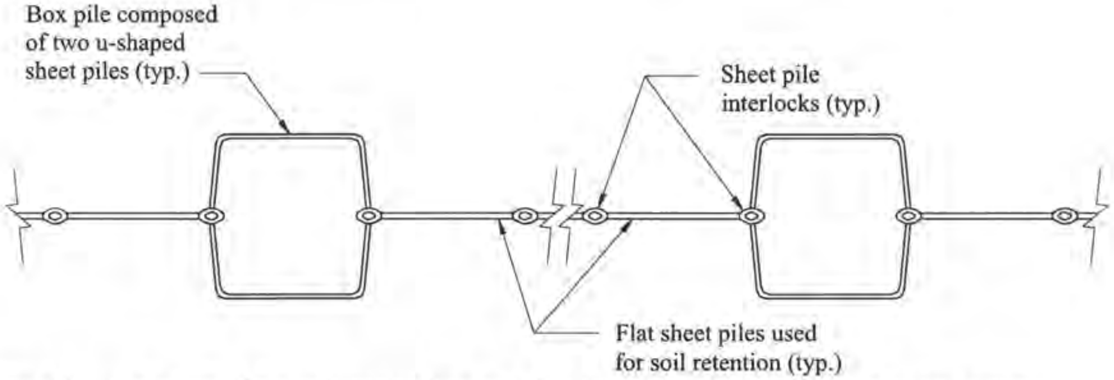


Figure 5.8. Plan view of a combination box and flat sheet pile abutment below the pile cap.

the superstructure caused by vehicle loads [52]. Sheet piles can be used as the wall facing for a GRS bridge abutment thus providing the scour protection for the GRS mass.

The Sprout Brook Bridge in Paramus, New Jersey highlights many of the advantages previously listed for a sheet pile abutment. The new 48 ft single span bridge was built in 1998 with a roadway width of 209 ft (13 traffic lanes). The original abutment design consisted of driven piles with a cast-in-place reinforced concrete pile cap built behind sheet pile cofferdams. An alternative substructure system was proposed that included using sheet piles driven to bedrock as the main structural element and an additional row of sheet piles for lateral support. This alternative design not only eliminated the need for the cofferdams, but also reduced the construction time by ten weeks at a savings of $280,000. The reduced earthwork also eliminated four of the originally planned six traffic phases. The sheet piles were designed for an axial load of 15 kip per ft and a maximum bending moment of 45 ft-kips per foot [52].

Additionally, a consulting firm in Alaska has developed an open cell sheet pile abutment system. As shown in Figure 5.9, a series of 15 in. flat sheet piles are driven in a circular pattern with an approximate radius of 30 ft, depending on the roadway width. The term open cell is used because the structure is not a completely closed circle. About 25 percent of the cell sheet piles in the system are not installed, all of which are located below the approaching roadway. The sheet piles do not need be deeply embedded to obtain lateral stability, instead the back tail piles provide lateral support by acting as a friction anchor for the bearing piles directly below the superstructure. Not installing the remaining 25 percent of the sheet piles requires less material and yields lower construction tolerances. Installation and compaction of the backfill is also easier when compared to closed cells because equipment can be moved in and out of the structure without the use of a crane. Additionally, the rounded stream face allows for a larger flow area that correlates to a small span length and lower bridge costs [55].

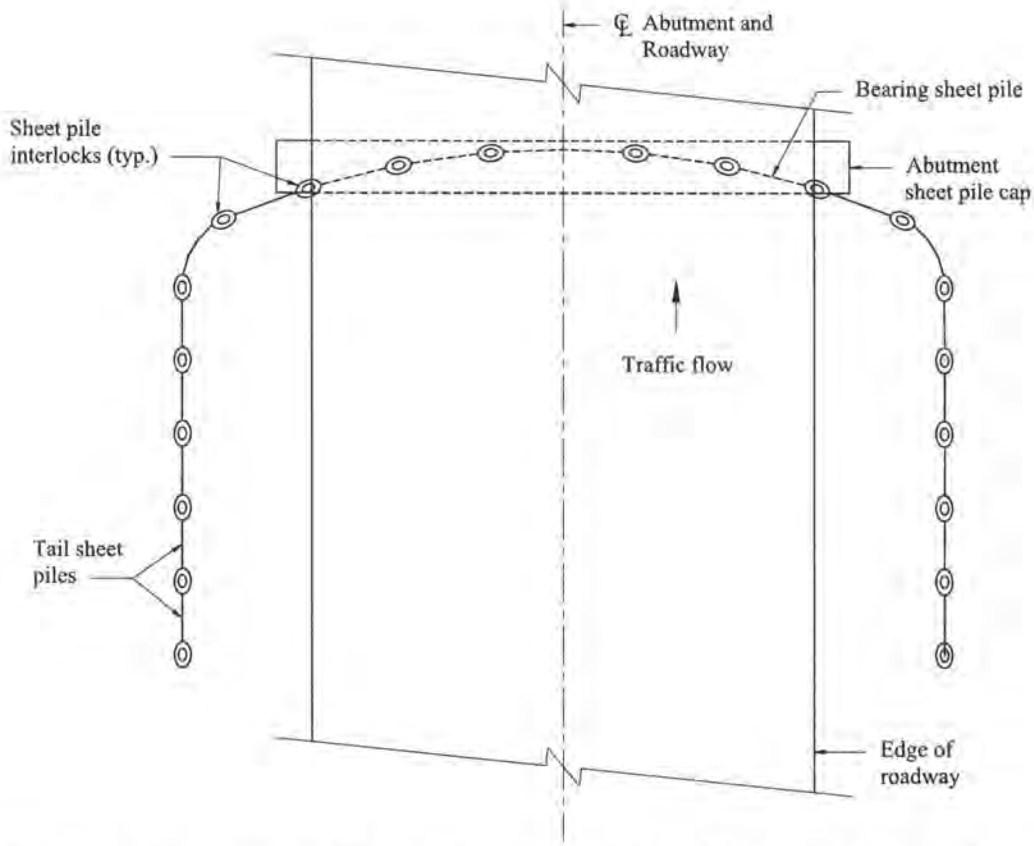


Figure 5.9. Plan view of an open cell sheet pile abutment [adapted from Nottingham et al., 2000].

6. REPORT SUMMARY

This research project consisted of three major phases: the collection of LVR bridge abutment information, the development of an abutment design methodology, and the creation of design aids for Iowa County Engineers, municipal engineers, etc. In the first phase, a literature review and survey of the Iowa County Engineers was completed in addition to the formation of the PAC. The literature review focused on locating LVR bridge abutment design information. The survey focused on the use of current knowledge and/or use of standard design sheets by the counties and the identification of common construction methods and trends. The PAC was composed Iowa County Engineers and representatives from the Iowa DOT Office of Bridges and Structures provided information relating to the scope of the project. This information included roadway and span length limitations, which superstructures should be accommodated by the standard abutment designs, a range of common backwall heights, and common pile material types. Additionally, members of the PAC suggested that if a flexible and easy to use design template is created (i.e., a spreadsheet or Visual Basic software), local Iowa Engineers would have more flexibility when designing an abutment. This phase of the project resulted in the identification of different LVR bridge abutment systems commonly used in Iowa counties, a series of alternative abutment systems, and two different pile analysis methodologies that could be used to investigate the influence of the lateral loading on the piles.

As previously mentioned, the literature search revealed several alternative abutment systems that are well established in a particular geographic region or for a specific use, however none of them have been used in Iowa bridge abutments. The alternative abutment systems include micropiles, GRS structures, Geopier foundations, and sheet pile abutments. Since these are economical and provide advantages over the traditional deep foundations systems currently used (i.e., driven piles), they show promise for numerous sites in Iowa. As noted in Chapter 7, it is proposed that several of these systems be investigated and tested in demonstration projects.

The second phase of this project involved investigating different lateral load analysis methodologies and the development of a foundation design methodology for the foundation elements. Two separate pile analysis methods were investigated, including a linear and a non-linear method. It was found that each method has certain advantages such as the ability to model complex soil conditions and profiles, accurately representing the actual soil and pile interaction, and the ease of incorporating the analysis method into a complete design methodology.

The maximum pile moments obtained from the linear and non-linear methods were compared; it was determined that the linear method is more conservative for most lateral load cases associated with LVR bridge abutments. For stiff cohesive soils, the linear method is more conservative by 7 to 15 percent depending on the magnitude of the lateral pile loadings. However, unlike stiff cohesive soils, the linear method produces less conservative maximum pile moment values in soft cohesive soils. The linear method is less conservative by about 3 to 20 percent depending on the magnitude of the lateral pile loading. Finally, the maximum pile moment values in cohesionless soils obtained from the linear method are more conservative by zero to three percent when compared to the non-linear analysis method.

Based on the relative simplicity and the correlation of the calculated maximum pile moment, it was decided that the linear analysis procedure presented by Broms [19, 20] would be the most suitable for this project. This method considers the pile fixed at a calculated depth below ground based on soil properties and lateral loading conditions. The maximum moment in the pile can then be calculated using basic structural analysis. The structural analysis procedure for the piles was developed using the recommendations of the AISC Manual of Steel Construction and the NDS for Wood Construction for steel and timber piles, respectively.

An analysis and design methodology was also developed for the lateral restraint system that can be used to resist the lateral substructure loads. Two lateral restraint systems are presented: a positive connection between the superstructure bearings and the substructure, and a buried concrete anchor connected to the substructure with the use of anchor rods. A positive connection between the superstructure bearings and substructure will transfer lateral loads between the superstructure units using the axial stiffness of the superstructure. The lateral restraint provided by an anchor system is a result of the passive soil pressure that acts on the vertical anchor face in the opposite direction of the lateral substructure loads as described by Bowles [12]. This lateral capacity is transferred to the pile system with the use of anchor rods and an abutment wale. The procedure for determining the structural capacity of the anchor block was developed using reinforced concrete design specifications in AASHTO.

The third and final phase of this project involved the development of the design aids that incorporate the previously mentioned design methodology. These design aids include a FDT with instructions and a series of generic standard abutment plans. The design spreadsheet is used to verify the adequacy of a pile and anchor system (if needed) for a particular bridge site. The engineer inputs data such as bridge geometry, soil conditions, pile information, and lateral restraint details. This information is used in an analysis of the foundation system to determine the capacity of the system, and to complete the required design checks. Finally, a series of generic standard abutment plans were created for different situations. This includes different standard sheets for each combination of steel or timber piles either with or without concrete anchors, a steel channel or concrete pile cap, and a backwall consisting of either timber planks or vertically driven sheet piles. The standard abutment sheets can be used by Iowa County Engineers to produce the necessary drawings for the more common LVR bridge abutments systems. In order for the engineer to produce a finished set of abutment construction sheets, the necessary details such as the bridge geometry, member size designations (i.e., W, C, and HP shapes), and material properties must be completed in the spaces provided. Volume II (i.e., Appendix C) of this report is a design manual for LVR bridge abutments that also presents the previously mentioned design aids in detail.

7. RECOMMENDED RESEARCH

Additional research is recommended to investigate other types of abutments mentioned in this report. A brief description of several items that should be investigated is presented below:

- The literature search revealed several alternative abutment systems that could be economical at certain sites. These systems include micropiles, GRS structures, Geopier foundations, and sheet pile abutments. Some of these systems are well established in certain regions of the country or for a specific use; however, none have been used in a bridge abutment system in Iowa. Since these appear to be economical and provide advantages over the traditional deep foundations systems currently used (i.e., driven piles), they show promise for use in Iowa. Thus, demonstration projects employing each of these four systems should be undertaken. Each of these abutment systems should be instrumented and monitored for at least three years. Design methodologies and generic plans (similar to those developed in this project) should be developed to assist engineers with their design.

- The use of precast substructure elements for bridge abutment should be investigated. A precast system has several advantages. An offsite precast yard is typically designed for faster production and better quality control when compared to onsite concrete construction. Thus, precast elements will result in a faster construction sequence of the bridge substructure and also result in a less congested construction site. Additionally, once the casting elements have been purchased, the overall cost of future abutments will be reduced. Demonstration projects using precast elements will document these advantages. The creation of standard details associated with the precast substructure elements presented below will also allow for easy distribution to the local engineers:

    - Pile cap that can also be used as a backwall (similar to Figures 3.2 and 3.3).
    - Wingwalls.
    - Backwall panels that are placed between the exposed steel H-piles (Figure 3.4).
    - Tieback systems (grouted in place)
    - Complete backwall systems that are post-tensioned to a system of piles.

- As previously noted, the design methodology and design aids developed in this investigation provide engineers with the tools to significantly reduce the time and effort required to design a LVR bridge abutment. Although the information and tools provided with this report can be applied to LVR bridge abutment design following the guides presented herein, a short course should be developed and administered to familiarize engineers the with the design methodology and design aids. This one-half day short course could be presented in each of the six transportation districts to minimize travel time for Iowa County Engineers.

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APPENDIX A

TR-486 SURVEY

Iowa Department of Transportation
Highway Division
Research Project TR-486

"Development of Abutment Design Standards for Local Bridge Designs"

Questionnaire completed by: ______________________________

Organization: ______________________________________

Address: ____

____

____

E-mail address: ______________________________________

Responses can either be E-mailed or faxed to F. W. Klaiber (E-mail address: klaiber@iastate.edu; Fax number: 515-294-7424). If you have some abutment designs, pictures, etc. that you are willing to share, please mail them to:

Prof. F. Wayne Klaiber, P.E.
422 Town Engr. Bldg.
CCEE Dept.
Iowa State University
Ames, Iowa 50011

Section 1

Q-1) Does your county have standard bridge abutment designs that are used on low-volume road bridges or off-system bridges.

Yes ______ No ______

If you answered no to Q-1, please skip the remaining questions (Q-2 – Q-6) in this section and complete the questions in Section 2.

Q-2) Would you please send us a copy of your standard abutment design(s).

Yes ______ No ______

Q-3) In what situations (conditions) are your standard abutment designs not applicable?

____

____

____

____

Q-4) What is the maximum superstructure span length used with your abutment standards?

L_{MAX} = _______________

Q-5) What type of construction equipment, special tools, etc. are required to install your standard abutments? Please indicate after each item if you own the equipment (O) or rent the equipment (R).

____

____

____

____

Q-6) Approximately how long does it take to install one of your standard abutments? ________ hours. Approximately how many workers are required to construct a standard abutment? ________________.

If you prefer, you can respond to Q-6 in man hours.

Section 2

Q-7) Do you know of other counties, cities, or other agencies that have standard abutment designs for low volume road bridges or off system bridges? If yes, please identify.

____

____

____

Q-8) Do you have a bridge construction crew that you routinely use to build small bridges or do you typically hire a contractor?

____

____

____

____

Q-9) Do you do a site investigation before installing substructures?

Yes _______ No _______

Q-10) If you do site investigations, what type (and number) of soil tests are completed?

____

____

____

____

Q-11) What types and how many foundation elements do you typically use in an abutment? How deep are they installed?

____

____

____

Comments?

____

____

____

____

____

APPENDIX B

TR-486 SURVEY SUMMARY

Table B.1. Summary of survey TR-486.

NOTE: NR means no response

Iowa DOT Transportation District	TR-486 Question 1	2	3	4	5	6	7
1	N	-	-	-	-	-	N
	N	-	-	-	-	-	N
	N	-	-	-	-	-	N
	N	-	-	-	-	-	Hardin and Jasper Counties
	N	-	-	-	-	-	N
	N	-	-	-	-	-	N
	N	-	-	-	-	-	N
2	N	-	-	-	-	-	N
	Y	Y	Not suitable when de-icing salts are used	40 ft	Crane, excavator, and sheet pile driver	96 man hours	N
	N	-	-	-	-	-	N
	N	-	-	-	-	-	Iowa DOT
	N	-	-	-	-	-	N
	N	-	-	-	-	-	N
	N	-	-	-	-	-	Iowa DOT
3	N	-	-	-	-	-	N
	N	-	-	-	-	-	N
	N	-	-	-	-	-	N
	N	-	-	-	-	-	N
	N	-	-	-	-	-	N
	N	-	-	-	-	-	Iowa DOT, Oden Enterprises
	N	-	-	-	-	-	N
	N	-	-	-	-	-	N
	N	-	-	-	-	-	N
4	N	-	-	-	-	-	N
	N	-	-	-	-	-	N
	Y	Y	Works for most conditions	20 to 40 ft	Crane or dragline, grader, bulldozer, and welder	3 - 6 workers, 3 - 4 weeks	N
	N	-	-	-	-	-	N
	N	-	-	-	-	-	N
	Y	Y	Shallow limestone or bedrock	40 ft	Dragline, pile driver, and backhoe (all owned)	200 - 240 man hours	N
	Y	Y	NR	NR	NR	NR	N
	N	-	-	-	-	-	N
	N	-	-	-	-	-	N
	Y	Y	Works for most situations	80 ft	Crane, pile driver, and sheet pile follow block (all owned)	160 man hours	N

Table B.1. Continued

NOTE: NR means no response

Iowa DOT Transportation District	TR-486 Question (continued) 8	9	10	11
1	Contractor	Y	Soil borings.	NR
	Contractor	Y	Soil report for nearest location is used	Depends on bridge site
	Contractor	N	Old records are used to estimate pile depths.	Mostly timber piles are used, however concrete and HP's are also used
	Contractor	Y	Soil borings at each abutment.	Standard abutment designs are typically used with typical pile depths of 25 and 60 ft for timber and HP's respectively
	Contractor	NR	NR	NR
	Crew	N	-	NR
	Contractor	Y	Four soil borings per bridge site.	HP's
2	Contractor	Y	One soil boring per abutment.	HP10 X 42, and 14 in. square precast concrete piles, typically 30 to 60 ft deep
	Crew	Y	Soil borings to 60 ft (or auger refusal), SPT test, soil classification.	At least five HP10 X 42's depending on span length, skews angles, and backwall height
	Crew	Y	One soil test per abutment (if anything).	Stub abutments approximately three to four feet deep
	Contractor	Y	One soil boring per abutment, soil classification, SPT test.	Mainly use Iowa DOT abutment standards, concrete integral abutments with steel HP's driven to refusal
	Contractor	Y	At least one soil boring per abutment, SPT test.	HP's are driven to bedrock or precast concrete piles are driven to glacial till
	Crew	Y	One soil test per abutment (if anything).	Stub abutments approximately three to four feet deep
	Contractor	Y	Depends on bridge geometry.	HP's driven to refusal and some timber friction piles, spread footings are rarely used
3	Contractor	Y	Soil borings to at least 50 ft.	Concrete, steel, and timber piles typically 30 to 60 ft deep
	Contractor	Y	SPT test at each abutment.	Six to eight timber piles averaging 35 ft deep
	Crew	N	-	Five or six HP10 X 42's averaging 30 ft deep
	Contractor	Y	Soil borings.	Foundation work is not done in house
	Contractor	Y	Four soil borings per bridge.	Timber, HP's and concrete filled pipe piles typically 30 to 80 ft deep
	Contractor	N	-	Oden Enterprises and Iowa DOT standard abutments are used
	Contractor	Y	Two to four soil borings depending on the number of spans.	HP's typically 45 to 60 ft deep
	Contractor	Y	Perform visual inspection or use soil borings.	HP depth determined by wave equation (blow count)
	Contractor	Y	SPT test at each abutment.	Six to eight timber piles typically 30 to 35 ft deep
4	Both	N	-	Seven concrete filled pipe piles typically 20 ft deep
	Contractor	Y	Consultant performs site investigation and provides recommendations.	Depends on bridge site
	Crew	N	Foundation design based on other bridge sites in the area.	Five or six HP's typically 40 to 50 ft deep
	Contractor	Y	Up to six soil borings.	HP's driven to bedrock
	Crew	N	-	Five to seven HP10's or 12's are used
	Crew	Y	At least one soil boring per abutment.	HP10 X 42's and timber piles typically driven to bedrock
	Both	Y	Test pile is driven (if anything).	HP10 X 42's driven to bearing
	Both	Y	Soil borings for larger bridge sites.	Treated timber piles are used
	Crew	N	-	Five to seven HP10 X 42's driven to refusal (typically about 60 ft deep)
	Crew	N	-	Five or six HP10 X 42's driven to a bearing of 17 to 20 tons

Table B.1. Continued

NOTE: NR means no response

Iowa DOT Transportation District	TR-486 Question (continued) Comments
	Estimating the pile depth from old records is cheaper than site investigation, bridges are designed by consultant
1	Would like to see high concrete abutment standard, exposed timber piles are not recommended
	In favor of standard abutment designs
	Would like to see a standard backwall that can be adapted for a different number of piles and span lengths
	The number of piles are determined by lateral and gravity loads.
	Standard drawings for high concrete abutments would be useful.
2	Would like to see standard plans for an integral abutment for 24 and 30 ft roadway widths
	In favor of standard abutment designs
	Mostly use Iowa DOT standards
	Standard abutment designs would be useful. Can try a precast concrete or sheet pile backwall.
	Oden Enterprise standard abutments are used, the ENR formula is used to determine bearing resistance
	Pile length is estimated using soil borings
3	
	Typically uses Iowa DOT slab bridge standards, would like to see the creation of standard that are easier to build
	Would like to see standard designs for high, stub, and fixed integral abutments. Does not recommend timber abutments
	Cass County does not have abutment standards but a common design theory is used
	In favor of standard abutment designs
	Formation of a bridge construction crew is in progress
4	Construction and cost limitations require the use of a contractor
	In favor of standard abutment designs

Table B.1. Continued

NOTE: NR means no response

Iowa DOT Transportation District	TR-486 Question 1	2	3	4	5	6	7
5	N	-	-	-	-	-	Davis County
	Y	Y	shallow bedrock	70 ft	Crane, pile driver, welder, and excavator (all owned)	120 man hours	Oden Enterprises
	N	-	-	-	-	-	N
	N	-	-	-	-	-	Warren County
	N	-	-	-	-	-	N
	N	-	-	-	-	-	Lucas County
	N	-	-	-	-	-	Guthrie County
	N	-	-	-	-	-	Davis County
	N	-	-	-	-	-	Decatur County
	N	-	-	-	-	-	N
6	Y	-	Not suitable when span length is increased	40 ft	Crane and pile driver	4 laborers	N
	N	-	-	-	-	-	N
	Y	Y	Skewed bridges, multiple spans	56 ft	Dragline or pile driver (owned)	72 man hours	Scott County

Table B.1. Continued

NOTE: NR means no response

Iowa DOT Transportation District	TR-486 Question (continued) 8	9	10	11
5	Contractor	N	-	HP10 X 42's typically 40 or 50 ft deep
	Crew	N	-	Seven HP 10 X 42's originally 40 ft in length and spliced if needed
	Contractor	N	-	Only HP standard plans are used
	Contractor	N	-	Timber piles typically 20 to 40 ft deep
	Contractor	Y	One soil test per abutment.	Mostly timber piles are used, however HP's are also used
	Contractor	N	-	HP10 x 42's driven to a bearing of at least 25 tons
	Contractor	Y	One soil boring per abutment, SPT test.	Timber and HP's of varying depth
	Contractor	N	-	HP10 X 42's typically about 40 to 50 ft deep
	Contractor	N	-	HP's driven to bedrock with a large range in depth
	Contractor	Y	Soil borings.	Five to nine piles typically 30 to 90 ft deep
6	Both	NR	NR	Four or five HP10 X 42's averaging 35 ft deep
	Crew	N	-	Five or six timber piles typically 35 ft deep
	Crew	N	-	Eight to ten inch diameter timber piles typically 35 ft deep

Table B.1. Continued

NOTE: NR means no response

Iowa DOT Transportation District	TR-486 Question (continued) Comments
5	In favor of standard abutment designs
	In favor of standard abutment designs
	Knows standard abutment plans exists but does not know who created them
	Approximately 14 years ago, Guthrie County had standard abutment designs
	In favor of standard abutment designs
	Mostly use Iowa DOT standards
6	In favor of standard abutment designs
	In favor of standard abutment designs

APPENDIX C

USERS MANUAL FOR LOCAL BRIDGE ABUTMENTS

The preceding Chapters 1 through 9 and Appendices A and B are similar to Volume I of the Iowa DOT Project TR-486 final report. A similar version of this Appendix C is published as Volume II of the same project. Therefore Appendix C of this thesis has its own Table of Contents, chapters, appendices, etc.

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1. INTRODUCTION

Various superstructure design methodologies have been developed by the Iowa State University (ISU) Bridge Engineering Center (BEC). However, to date no standard abutment designs or design methodologies had been developed. Obviously, with a set of standard abutment plans and the various superstructures systems, a county engineer could design the complete bridge for a given site. Thus, there was a need to establish an easy-to-use design methodology and standard abutment plans for the more common substructure systems used in the Iowa counties.

1.1. OBJECTIVE AND SCOPE OF ABUTMENT DESIGN AIDS

The objective of this project was to develop a simple design methodology, a series of standard abutment plans, and a series of design aids for the more commonly used substructure systems in Iowa counties. The design aids include: 1.) graphs for estimating dead and live load abutment reactions, 2.) a summary table of estimated allowable pile end and friction bearing values based on the Iowa Department of Transportation Foundation Soil Information Chart (Iowa DOT FSIC) [1], 3.) a generic foundation design template (FDT), and 4.) generic standard abutment plans. When used correctly, these tools will assist the Iowa County Engineers in the design and construction of low-volume road (LVR) bridge abutments.

The assumptions incorporated in the developed design methodology and corresponding design aids are similar to those made for a stub abutment system. The applicability of the design aids are limited to span lengths ranging from 20 to 90 ft and are intended for roadway widths of 24 and 30 ft (however abutments for other roadway widths can be designed with the FDT). Also, the soil profile must be relatively uniform and mostly consist of a cohesive or cohesionless soil. Superstructure systems other than the beam-in-slab bridge (BISB), railroad flat car (RRFC), pre-cast double tee (PCDT), glued-laminated girders (glulam), prestressed concrete (PSC), quad-tee, and slab bridge systems are not incorporated in the LVR bridge abutment design aids. However, the general design methodology can, in theory, be applied to the design of substructures for a variety of other superstructure systems.

1.2. REPORT SUMMARY

As previously stated, a similar version of this thesis is published in two volumes as the Iowa Department of Transportation (Iowa DOT) Project TR-486 final report [2, 3]. Chapters 1 through 9 and Appendices A and B of this thesis is similar to Volume I whereas Appendix C is similar to Volume II and therefore has its own Table of Contents, chapters, appendices, etc. Chapters 1 through 9 and Appendices A and B of this thesis will be referred to as Volume I and Appendix C will be referred to as Volume II herein. All appendices referenced herein refer to Volume II unless otherwise noted.

Volume I (i.e., Chapters 1 through 9 and Appendices A and B): *Development of Abutment Design Standards for Local Bridge Designs* provides a summary of the tasks completed in the project. This includes a survey of the Iowa County Engineers, the collection of input from a Project Advisory Committee (PAC), the development of a LVR bridge abutment design methodology, and a summary of research required to expand the types of abutments that could be used on LVR bridges.

Volume II (i.e., this Appendix C): *A Design Manual for Local Bridge Abutments* provides instructions for using the previously mentioned design aids. This includes a detailed description of all required input parameters for the FDT, a description of the design requirements, and recommendations for optimizing the pile and anchor system to effectively meet these requirements. Instructions for using the estimated gravity load charts, estimated pile bearing tables, and standard abutment plans are also included in this volume. Additionally, design verification examples which demonstrate the application of the design methodology and the foundation design template were completed but are not included herein.

2. DESIGN METHODOLOGY SUMMARY

A brief summary of the design methodology developed for LVR bridge abutments is presented in this chapter. This includes the determination of the substructure loads, the structural analysis, foundation capacity calculations, and checking design requirements for the pile and anchor systems. Additional substructure elements such as the pile cap, abutment wale, and backwall also need to be investigated; however, a design methodology for these elements was beyond the scope of this project. A graphical representation of the design methodology summarized herein is shown as Figure 4.1 of Volume I and is included here as Figure 2.1.

2.1. DESIGN LOADS

The first step in designing a foundation is the identification of loads. Gravity loads include the bridge self-weight in addition to live loads. Lateral loadings are imparted to the bridge substructure by active and passive soil pressures in addition to lateral forces transmitted from the superstructure to the substructure through the bridge bearings.

2.1.1. Gravity Loads

Gravity loads include the self-weight of the bridge roadway surface, superstructure, and substructure in addition to bridge live loads. Conservative dead load abutment reactions for PCDT, PSC, quad tee, glulam, and slab bridge systems are given in Figures A.1 and A.2 of Appendix A for 24 and 30 ft roadway widths, respectively. These estimates are based on published standard bridge designs for the respective superstructure systems and include the self weight of both the superstructure and substructure. More accurate, and potentially smaller, dead load abutment reactions can be determined using site-specific bridge information. The dead load abutment reactions for other standard superstructure systems such as the RRFC and BISB systems are not provided since there are numerous cross sections available and thus there are significant variations in the self weight.

The live load abutment reaction is computed using the American Association of State Highway Transportation Officials (AASHTO) Standard Specifications for Highway Bridges, Sixteenth Edition [4] HS20-44 design truck. Additional loads such as the AASHTO lane load and Iowa legal loads were also investigated, however the HS20-44 truck controls for all span lengths within the scope of this project (i.e., between 20 and 90 ft). The maximum simple span abutment reaction occurs when the back axle is placed directly over the centerline of the piles with the front and middle axles on the bridge. The live load abutment reactions for two, 10 ft wide design traffic lanes without impact are provided in Figure A.3 of Appendix A. These values can be proportioned for a different number of design traffic lanes depending on the roadway width.

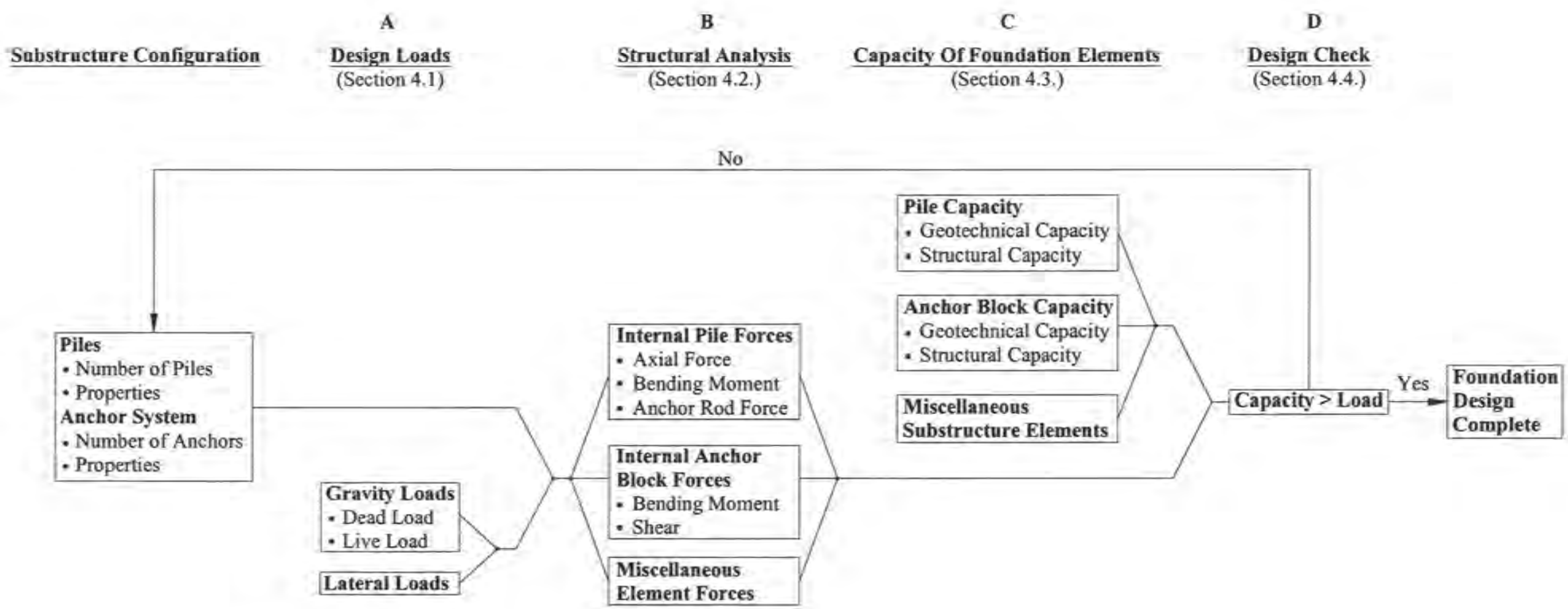
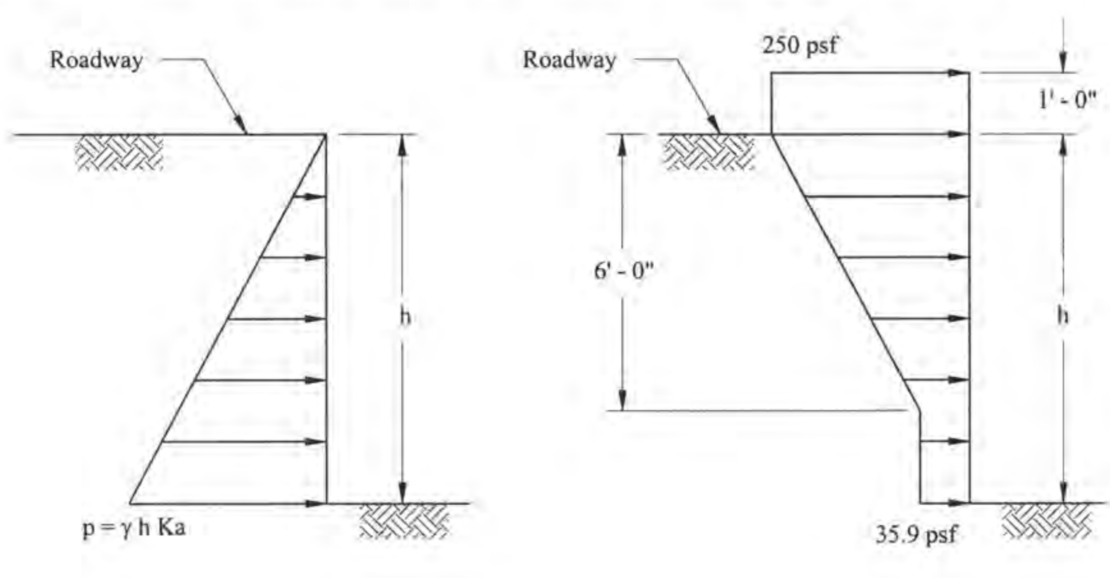


Figure 2.1. Graphical representation of the design methodology for a LVR bridge abutment [Figure 4.1 of Volume I].

2.1.2. Lateral Loads

The substructure systems commonly used by Iowa counties require the piles to resist lateral loads in addition to gravity loads. The Iowa Department of Transportation Bridge Design Manual (Iowa DOT BDM) [5] specifies two different horizontal soil pressures for bridge substructures as shown in Figure 2.2. The first pressure distribution (Figure 2.2a) represents the active soil pressure attributed to the permanent loading of the backfill soil. The second pressure distribution (Figure 2.2b) represents a gravity live load on the approach roadway. This live load is modeled as an equivalent soil surcharge equal to two feet of soil above the approach roadway thus resulting in the pressure distribution shown. Both lateral soil pressure distributions are included in the design methodology for this project.

Other lateral bridge loadings such as longitudinal wind forces, transverse wind forces, and a longitudinal braking force are also listed in the Iowa DOT BDM. Longitudinal wind forces were investigated and found to be negligible for LVR bridge abutments and therefore were not included in the design methodology for this project. The longitudinal braking force is equal to five percent of the total gravity component for the AASHTO lane loading multiplied by the number of 10 ft design lanes. One type of transverse wind load consists of a 50 psf pressure that acts on the elevation surface area of the superstructure, roadway and barrier rail. This transverse loading acts perpendicular to the flow



a) Active soil pressure distribution.

b) Equivalent live load surcharge.

Figure 2.2. Lateral soil pressure distributions [adapted from the Iowa DOT BDM, 2004; Figure 4.5 of Volume I].

of traffic. A second transverse wind load, also perpendicular to the flow of traffic, consists of a 100 plf line load that represents wind acting on the bridge live load. Both transverse wind loads and the longitudinal braking force were included in the design methodology for this project. The load groups cited in Section 6.6 of the Iowa DOT BDM are used to determine the maximum loading effects for various combinations of gravity and lateral loadings.

2.2. STRUCTURAL ANALYSIS

Once the substructure loads have been determined, a structural analysis of the foundation system can be performed to determine the internal forces. This includes the pile axial force and bending moment, anchor rod axial force, and the internal anchor block shear and bending moment.

2.2.1. Internal Pile Loads

The total abutment reaction, which is the sum of the dead and live load abutment reactions, is used to determine the individual axial pile forces. The axial pile loads (i.e., the load each pile much resist) are a function of the total number of piles and their spacing plus the superstructure bearing points. Different combinations of pile and superstructure bearings point configurations will produce various maximum axial pile forces within a given pile group. Therefore, a nominal axial pile factor was developed using structural analysis software for all superstructure systems included with this design methodology to account for the different axial forces that can develop. The design axial pile force is equal to the total abutment reaction divided by the number of piles times the nominal axial pile factor shown in Table 2.1. As previously discussed, the total abutment reaction is the sum of the dead and live load reactions which are used to determine the individual axial pile load.

The lateral load analysis technique used in this design methodology is reported by Broms [6, 7]. Specifically, the pile is considered fixed at a calculated depth below ground and is analyzed as a cantilever structure. The depth to fixity is a function of different parameters such as the

Table 2.1. Nominal axial pile factors for various superstructure systems [Table 4.1 of Volume I].

Superstructure System	Nominal Axial Pile Factor
PCDT	1.40
BISB	1.35
RRFC (Type 1)	1.20
RRFC (Type 2)	1.40
Prestressed girder	1.30
Slab bridge	1.00
Quad-tee	1.50
Glulam girder	1.40

pile width and the above ground lateral pile loads. The undrained shear strength and friction angle of the soil are also required for cohesive and cohesionless soils, respectively.

A lateral restraint system can be used to reduce the lateral loading effects on the piles. The lateral restraint systems incorporated into the design methodology were a buried reinforced concrete anchor block connected to the substructure with tension rods, and a positive connection between the superstructure and substructure.

If a lateral restraint system is not utilized, the system is statically determinant and the maximum pile bending moment and deflection are found using statics. Superposition can be used to determine the combined effects of all the lateral pile loadings.

The incorporation of a lateral restraint system creates a statically indeterminate system. The structural analysis methodology for this project uses an iterative, consistent deformation approach in which the displacement of the lateral restraint system is equal to the displacement of the pile at the connection point.

2.2.2. Internal Anchor Block Forces

Once the anchor rod force per pile has been determined, the internal anchor block bending moment and shear loads can be calculated. The anchor force per pile, in addition to other parameters such as the elevation of the anchor, anchor rod properties, and pile spacing required for the structural analysis of the pile system are also used in the structural analysis of the anchor block.

The anchor block is analyzed as a continuous beam with simple supports that correspond to the locations of the anchor rods. The net soil reaction imparted on the anchor block to resist the lateral substructure loads is represented by a uniformly distributed load equal to the anchor rod force per pile, multiplied by the number of piles, and divided by the total length of the anchor block. The internal anchor block shear and bending moment can be determined using a number of indeterminate structural analysis techniques.

2.3. CAPACITY OF FOUNDATION ELEMENTS

The guidelines specified in the Iowa DOT BDM, AASHTO, the National Design Specification Manual for Wood Construction (NDS Manual) [8], and the American Institute of Steel Construction, Manual of Steel Construction (AISC Manual) [9] are all used to determine the capacities of the various foundation elements.

2.3.1. Pile Capacity

For this project, a foundation pile is classified as one of three different groups; end bearing piles, friction bearing piles, or combined friction and end bearing piles. The bearing capacity of an end bearing pile is attributed to the bearing of the pile tip on a relatively hard foundation material.

Appendix B contains estimated end bearing values for various H-pile sizes and foundation materials as cited by the Iowa DOT FSIC. The bearing capacity of friction piles is attributed to the shear forces developed between the embedded pile surface and the surrounding soil. The magnitude of this resistance varies significantly with both the pile and soil type. Appendix B also contains estimated friction bearing values for various pile and soil type combinations. The bearing capacity of a combined friction and end bearing pile is equal to the sum of the end bearing and friction bearing resistances previously described.

The Iowa DOT BDM states that piles are to be designed using allowable stress design methods. During the investigation of the different substructure design methodologies used by the Iowa County Engineers, it was discovered that specifications provided in the AISC Manual were used to investigate the structural capacity of steel piles subjected to both bending and axial loads. Therefore all equations used for the design methodology of steel piles in this section are taken from Sections 1.5 and 1.6 of the AISC Manual and are also provided in Appendix E. It should be noted that similar design specifications summarized in this section are also provided in AASHTO. As previously noted, the piles for typical LVR bridge abutments used by Iowa counties are required to resist both axial and bending forces. Therefore, interaction equations for steel piles subjected to combined loads are used.

The design capacity of timber piles are determined using the guidelines specified by AASHTO and the NDS Manual as summarized in Appendix E. The timber material properties vary significantly with the species type, member size and shape, loading conditions and surrounding environmental conditions. Therefore, timber modification factors are used to account for these variables. All modification factors used in the design methodology for timber piles are taken from AAHSTO, Section 13. As recommended by AASHTO, the interaction equation defined by the NDS Manual is used to verify the structural adequacy of timber piles subjected to combined axial and bending forces.

2.3.2. Anchor Block Capacity

The structural capacity of the anchor block in addition to the passive resistance of the surrounding soil must also be determined. The lateral capacity of the anchor system is related to the mobilized soil pressure that acts on the vertical anchor block face. The magnitude of the soil pressure is a function of the surrounding soil properties and the depth of the anchor block with respect to the roadway surface. The maximum lateral capacity of the anchor block (per pile) is determined by multiplying the passive soil resistance per foot by the pile spacing. The information used to determine the lateral capacity of the anchor system is cited in Bowles [10] and is provided in

Appendix E. Bowles also states that the maximum anchor efficiency is achieved when the anchor block is positioned beyond the passive and active soil failure planes behind the backwall face as shown in Figure 2.3.

Once the lateral capacity of the anchor block has been calculated, the structural capacity of the anchor block must be determined. The anchor block capacity is determined using reinforced concrete design practices as described in Section 8 of AASHTO. This includes the design of the flexural and shear reinforcement in addition to checking the flexural reinforcement development length, the ductility, and the minimum reinforcement requirements.

2.4. PILE AND ANCHOR SYSTEM DESIGN REQUIREMENTS

Once the internal forces and capacities have been determined, one must check the adequacy of the foundation system. In general, this consists of verifying that the individual element capacities are greater than the applied loads. For design bearing requirements, the capacity must be greater than the axial pile load. Additional requirements are cited by AASHTO and the Iowa DOT BDM. Due to the presence of combined bending and axial loads, the structural capacity of the pile is not directly determined. Rather, interaction requirements previously described are used to compare the ratios of applied to allowable stresses for combined bending and axial loadings. If the interaction equations yield a value less than 1.0, the pile is structurally adequate. However, if this requirement is not satisfied, an alternative pile configuration and corresponding loads must be used.

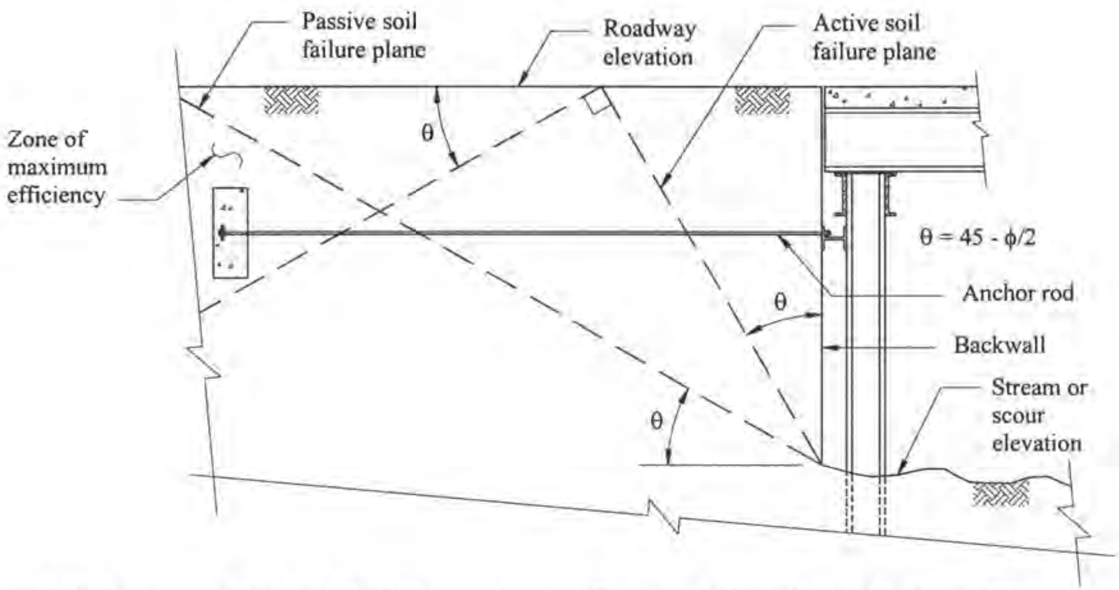


Figure 2.3. Location of anchor block for maximum efficiency [adapted from Bowles, 1996; Figure 4.6 of Volume I].

The capacity of the anchor system must also be verified. The applied anchor rod stress must be less than the allowable anchor rod stress defined in the AISC Manual. The maximum lateral capacity of the soil surrounding the anchor block (per pile) must be greater than the required anchor force per pile. In order to satisfy the structural design requirements, the internal anchor block shear and bending forces must be less the structural capacity of the anchor block determined using AASHTO reinforced concrete design guidelines.

3. DESIGN AID INSTRUCTIONS

This chapter provides the instructions for using the various low-volume road (LVR) bridge abutments design aids developed for this project. These design aids include: 1.) graphs for estimating dead and live load abutment reactions 2.) estimated pile end bearing and friction bearing values, 3.) the FDT, and 4.) generic standard abutment plans.

3.1. ESTIMATED GRAVITY LOADS

The estimation of both dead and live load abutment reactions based on various superstructure systems, span lengths, and roadway widths are presented in Appendix A. Conservative dead load abutment reactions for PCDT, PSC, quad tee, glulam, and slab bridge systems are shown in Figures A.1 and A.2 for 24 and 30 ft roadway widths, respectively. More accurate and potentially smaller dead load abutment reactions can be determined using site-specific bridge information. The live load abutment reactions without impact for two AASHTO HS20-44 design trucks are shown in Figure A.3. Data from Figure A.3 can be proportioned for a different number of design traffic lanes as needed. However if more than two traffic lanes are considered, the lane reduction factor specified in Section 3 of AASHTO (i.e., 0.90 and 0.75 for three and four or more traffic lanes, respectively) should be multiplied by the proportioned gravity live load.

To obtain the dead load abutment reaction, use either Figure A.1 or A.2, the bridge span length, and the line representing the appropriate bridge superstructure system. The live load abutment reaction is determined in the same manner using Figure A.3.

3.2. FOUNDATION DESIGN TEMPLATE

The FDT is used to verify the design of a given foundation system. At most, there are two worksheets that the engineer will be required to use. These include the Pile Design and Anchor Design worksheets (PDW and ADW, respectively). The use of the ADW may not be necessary depending on the bridge site. In the complete FDT, there are four different PDW, one for each combination of pile type (steel or timber) and soil type (cohesive or cohesionless). The engineer is automatically re-directed to the appropriate PDW by the clicking the appropriate button on the Start worksheet of the FDT (see Figure 3.1). It should be noted that the BEC logo in Figure 3.1 and applicable subsequent figures can be replaced with the logo of the county or consulting firm.

A numbering system is used to correlate the input values in the FDT with the descriptions provided in this chapter. Many input values, such as the roadway width, number of piles and lateral material property are required for both pile types. Therefore, the instructions for using the FDT for

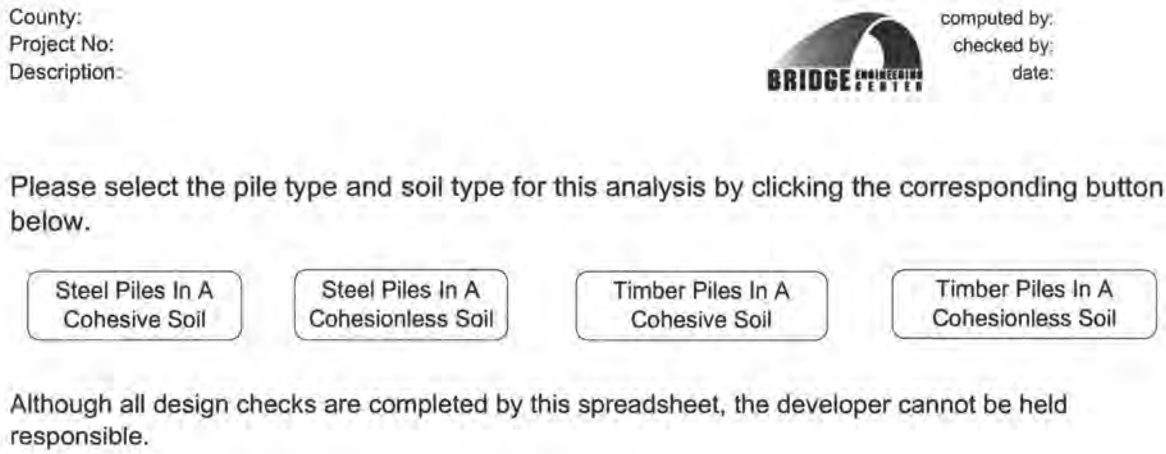


Figure 3.1. View of the Start worksheet for the FDT.

steel and timber piles are separated into three sections: 1.) steel piles in a cohesive or cohesionless soil, 2.) timber piles in a cohesive or cohesionless soil, and 3.) anchor block design. The instructions for using the ADW are applicable to all combinations of piles and soil types. Printouts of all worksheets produced by the FDT for each combination of pile and soil type are located in Appendix C. In the case where a subsurface bridge site investigation reveals a non-uniform soil profile consisting of both cohesive and cohesionless soils, the properties of the upper level soil should be used to determine which PDW should be used.

3.2.1. Steel Piles in a Cohesive or Cohesionless Soil

3.2.1.1. INSTRUCTIONS WORKSHEET

The Instruction Worksheet (IW) provides a brief description of the input quantities required for the PDW. A portion of the IW for steel piles is shown in Figure 3.2. Also, the IW contains a figure of an abutment cross section and roadway cross section near the abutment which is reproduced in Figure 3.3. This figure provides a graphical representation of some of the required input values. Each circled number in Figure 3.3 corresponds to an input cell number on the IW and PDW for steel piles (Figures 3.2 and 3.4, respectively). Once the IW has been reviewed, the engineer may proceed by clicking the 'PDW' button (in the upper left corner as shown in Figure 3.2).

3.2.1.2. REQUIRED INPUT

This section provides a detailed explanation of the input values required for the PDW for a steel pile. As shown in Figure 3.4, each input cell is highlighted. The quantities shown in the highlighted input cells of Figure 3.4 are not applicable for all bridge sites and are shown for

County:
Project No:
Description:

computed by:
checked by:
date:

The calculations performed in the Pile Design Worksheet are based on the guidelines of the AASHTO Standard Specifications, the AISC Manual of Steel Construction, and the Iowa DOT Bridge Design Manual (Iowa DOT BDM).

Once the instructions in this worksheet have been reviewed, proceed to the Pile Design Worksheet or return to the pile and soil selection worksheet by clicking the icons below.

Pile Design Worksheet

Return to Pile and Soil Selection Worksheet

Data required is to be entered in the highlighted cells of the Pile Design Worksheet.

The stream elevation is the datum for all elevations.

The following numbers and explanations correspond the highlighted cells on the Pile Design Worksheet; all circled numbers are shown on the figure provided.

Cell No.	Description
1	Enter the span length between the centerline of the abutment bearings.
2	Enter the roadway width of the bridge.
3	Enter the distance between the centerline of the exterior pile and the edge of the roadway. This value is positive for situations when the exterior pile is within the limits of the roadway width as shown above.
4	Enter the number of piles per abutment. This value must be within the range given in the cells directly above this input cell.
5	Enter the vertical distance between the roadway grade and the stream elevation.
6	Enter the vertical distance from the stream elevation to the estimated depth of scour. This value is based on stream hydraulics, geological information, and engineering judgment.
7	Use the pull-down menu provided to select the type of superstructure system for this analysis.
8	Enter the dead load abutment reaction for this analysis. A default value may be provided in the cell directly above this input cell.
9	Enter the live load abutment reaction for this analysis. A default value is provided directly above this input cell.
10	Enter the average standard penetration test (SPT) blow count (N-value) for the upper level soil.
11	Enter the undrained shear strength of the soil for this analysis. A default value based on the SPT N-value is provided in the cell directly above this input cell.
12	Use the pull-down menu provided to select the type of pile bearing resistance for gravity loads. NOTE: End bearing is only allowed in bed rock for this spreadsheet.
13	If applicable, enter the friction bearing resistance per foot of pile, for the soil *within* 30 ft of the natural ground line. Appendix C of Volume I provides friction bearing values based on the SPT N-value.
14	If applicable, enter the friction bearing resistance per foot of pile, for the soil *not within* 30 ft of the natural ground line. Appendix C of Volume I provides friction bearing values based on the SPT N-value.
15	If applicable, enter the estimated depth to adequate end bearing foundation material.
16	If applicable, use the pull-down menu provided to select the SPT N-value for the end bearing foundation material.
17	Use the pull-down menu provided to select the pile yield stress.
18	Use the pull-down menu provided to select an H-pile shape. If a standard shape is selected, input values for cell 19 through 25 will not be required from the engineer.
19	If applicable, enter the cross-sectional area of the pile.
20	If applicable, enter the pile width measured parallel to the backwall.

Figure 3.2. Selected portion of the FDT IW for a steel pile.

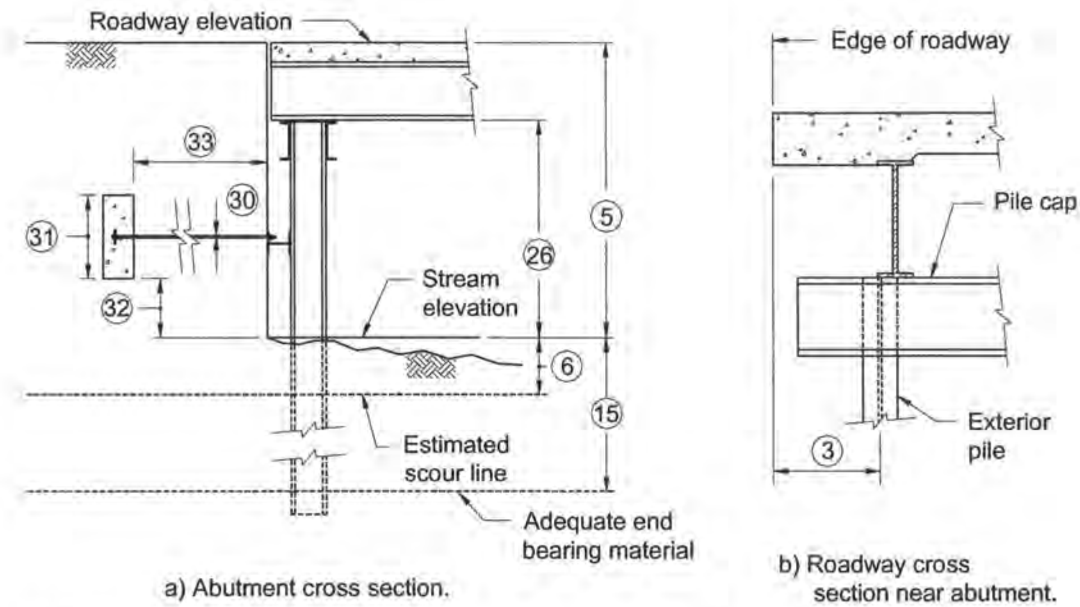


Figure 3.3. Graphical representation of selected input variables for steel piles.

demonstration purposes only. The only difference between the PDW for steel piles in a cohesive or cohesionless soil is the required soil input parameter (undrained shear strength and soil friction angle, respectively).

1. Span length (ft) – Enter the bridge span length as measured from the centerlines of the bridge abutments. This input value is limited to a value between 20 and 90 ft.
2. Roadway width (ft) – Enter the bridge roadway width. This input value must be greater than or equal to 24 ft.
3. Location of the exterior pile relative to the edge of the roadway (ft) – Enter the horizontal distance, 3, between the centerline of the exterior pile and the roadway edge as shown in Figure 3.3b. This value, limited to plus or minus 5 ft, is positive if all piles are located within the exterior limits of the roadway as shown in Figure 3.3b.
4. Number of piles (no units) – Enter the number of piles. This value must be a whole number that falls within the ranged specified in the two cells located directly above this input cell. The range of piles provided is based on the roadway width, location of the exterior pile relative to the edge of the roadway (input Cells 2 and 3, respectively), and spacing limitations cited in Section 6.2.4 of the Iowa DOT BDM.

County:
Project No:
Description:

computed by:
checked by:
date:

Instructions Worksheet

Go to Pile and Soil Selection Worksheet

General Bridge Input	1	Span length		60.00	ft	
	2	Roadway width		24.00	ft	
	3	Location of exterior pile relative to the edge of the roadway		0.75	ft	
		Maximum number of piles		10	piles on	2.50 ft centers
		Minimum number of piles		4	piles on	7.50 ft centers
	4	Number of piles	6			
	5	Backwall height		8.00	ft	
	6	Estimated scour depth		2.00	ft	
	7	Superstructure system	PCDT			
		Estimated dead load abutment reaction		180.9	kip per abutment (default value)	
	8	Dead load abutment reaction for this analysis		180.9	kip per abutment	
		Estimated live load abutment reaction		121.5	kip per abutment (default value)	
	9	Live load abutment reaction for this analysis		121.5	kip per abutment	
Foundation Material Input	10	Soil SPT blow count (N)		10		
		Correlated soil un-drained shear strength (C_u)		1,270	psf	
	11	Soil undrained shear strength for this analysis		1,270	psf	
	12	Type of vertical pile bearing resistance	friction & end bearing			
	13	Estimated friction bearing values for depths less than 30 ft		0.7	tons per ft	
	14	Estimated friction bearing values for depths greater than 30 ft		0.8	tons per ft	
	15	Depth to adequate end bearing foundation material		40	ft	
	16	SPT blow count for end bearing foundation material	100 < N < 200			
Pile Input	17	Pile yield stress		36	ksi	
	18	Select pile type	HP10x42			
	19	Pile cross sectional area		12.4	sq. in.	
	20	Pile flange width		10.1	in.	
	21	Pile moment of inertia (strong axis)		210	in.^4	
	22	Pile section modulus (strong axis)		43.4	cubic in.	
	23	Pile section modulus (weak axis)		14.2	cubic in.	
	24	Pile radius of gyration (strong axis)		4.13	in.	
	25	Pile radius of gyration (weak axis)		2.41	in.	
Lateral Restraint Input	26	Superstructure bearing elevation		5.00	ft	
	27	Type lateral restraint system	buried concrete anchor block			
	28	Anchor rod yield stress		60	ksi	
	29	Total number of anchor rods per abutment		6	per abutment	
	30	Anchor rod diameter		0.88	in.	
	31	Height of anchor block		2.50	ft	
	32	Bottom elevation of anchor block		3.00	ft	
		Anchor block geotechnical capacity		10.8	kip per pile	
		Computed anchor force per pile		9.7	kip per pile	
		Minimum anchor rod length		14.69	ft	
	33	Anchor rod length for this analysis		16.00	ft	

Check Pile Design

Figure 3.4. Input section of the FDT PDW for steel piles.

5. Backwall height (ft) – Enter the vertical distance, (5), between the stream elevation and roadway elevation as shown in Figure 3.3a.
6. Estimated scour depth (ft) – Enter the estimated depth of soil, (6), that could potentially be eroded away due to scour as shown in Figure 3.3a. This value should be based on hydraulic and geological information as well as engineering judgment.
7. Superstructure system (no units) – Use the provided pull-down menu to select the appropriate superstructure being used.
8. Dead load abutment reaction for this analysis (kips per abutment) – Enter the dead load abutment reaction for this analysis. If a 24 or 30 ft roadway width and a superstructure system other than a BISB and RRFC are used, a conservative value will be shown in the cell located directly above this input cell as shown in Figure 3.4. This default value is based on span length, roadway width, and the superstructure used (input Cells 1, 2, and 7, respectively).
9. Live load abutment reaction for this analysis (kips per abutment) – Enter the live load abutment reaction for this analysis. A conservative value is provided in the cell directly above this input cell as shown in Figure 3.4. This default value is based on the span length and roadway width (input Cells 1 and 2, respectively).
10. Soil SPT blow count (N) – Enter the SPT blow count for the soil in the immediate vicinity of the foundation piles. If a non-uniform soil profile is present, use the average blow count for the upper level soil. This input value must be a whole number between 1 and 50.
11. Soil undrained shear strength for this analysis, **for steel piles in cohesive soil only** (psf) Enter the undrained shear strength (c_U); a default value based on the most commonly used correlation of the SPT blow count and undrained shear strength as reported by Terzaghi and Peck [11] is provided in the cell directly above this input cell as shown in Figure 3.4. This relationship is provided as Equation 3.1. Since this correlation can be unreliable for some in-situ conditions, it is recommended that the undrained shear strength be determined by testing soil samples from the bridge site. This input value is used to calculate the depth of pile fixity for piles in cohesive soils, the equation for which is presented in Appendix E.

$$c_U = 0.06 * N * P_{ATM} \tag{3.1}$$

where:

c_U = Soil undrained shear strength.

N = SPT blow count.

P_{ATM} = Atmospheric pressure.

11. Soil friction angle for this analysis, **for steel piles in cohesionless soil only** (degrees)

    Enter the soil friction angle (ϕ); a default value based on a correlation of the SPT blow count and the soil friction angle as reported by Peck [12] is provided in the cell directly above this input cell. This input value is not shown in Figure 3.4 in lieu of the undrained shear strength. This relationship is provided as Equation 3.2. It is recommended that the soil friction angle be determined from tests on soil samples from the bridge site. This input value is used to calculate the depth of pile fixity for piles in cohesionless soils, the equation for which is presented in Appendix E.

$$\phi = 53.881 - \left(27.6034 * e^{-0.0147\,N}\right) \tag{3.2}$$

where:

N = SPT blow count.

ϕ = Soil friction angle.

12. Type of vertical pile bearing resistance (no units) – Use the provided pull-down menu to select an appropriate type of vertical bearing resistance.
13. Estimated friction bearing value for depths less than 30 ft (tons per ft) – If applicable, enter an estimated friction bearing resistance for the soil *within* 30 ft of the natural ground line. Estimated values for this input parameter can be obtained from Appendix B or the Iowa DOT FSIC. This input value must be between 0.1 and 2.0 tons per foot.
14. Estimated friction bearing value for depths greater than 30 ft (tons per ft) – If applicable, enter an estimated friction bearing resistance for soils *not within* 30 ft of the natural ground line. Estimated values for this input parameter can be obtained from Appendix B or the Iowa DOT FSIC. This input value must be between 0.1 and 2.0 tons per foot.
15. Depth to adequate end bearing foundation material (ft) – If applicable, enter the estimated depth below stream elevation to adequate end bearing foundation material, 15, as shown in Figure 3.3a. This input value must be greater than 10 ft as cited by the Iowa DOT BDM.
16. SPT blow count for end bearing foundation material (N-value) – If applicable, use the provided pull-down menu to select an estimated SPT blow count range for the end bearing foundation material.

17. Pile yield stress (ksi) – Use the provided pull-down menu to select the pile yield stress.
18. Select pile type (no units) – Use the provided pull-down menu to either select a standard H-Pile shape or the option to manually input the pile properties defined below for input Cells 19 through 24.
19. Pile cross sectional area (in.^{2}) – If applicable, enter the cross sectional area of the pile.
20. Pile flange width (in.) - If applicable, enter the pile width measured parallel to the backwall face.
21. Pile moment of inertia (in.^{4}) – If applicable, enter the strong axis moment of inertia. For this analysis, it is assumed that the strong pile axis is parallel to the backwall face.
22. Pile section modulus (in.^{3}) - If applicable, enter the *strong* axis section modulus. For this analysis, it is assumed that the *strong* pile axis is *parallel* to the backwall face.
23. Pile section modulus (in.^{3}) – If applicable, enter the *weak* axis section modulus. For this analysis, it is assumed that the *weak* pile axis is *perpendicular* to the backwall face.
24. Pile radius of gyration (in.) - If applicable, enter the *strong* axis radius of gyration. For this analysis, it is assumed that the *strong* pile axis is *parallel* to the backwall face.
25. Pile radius of gyration (in.) - If applicable, enter the *weak* axis radius of gyration. For this analysis, it is assumed that the *weak* pile axis is *perpendicular* to the backwall face.
26. Superstructure bearing elevation (ft) – Enter the vertical distance between the stream elevation and superstructure bearings, 26, as shown in Figure 3.3a. This input value must be between 0 ft and the backwall height (input Cell 5).
27. Type of backwall lateral restraint system (no units) – Use the provided pull-down menu to select the lateral restraint system for this analysis.
28. Anchor rod yield stress (ksi) – If applicable, use the pull down menu provided to select the anchor rod yield stress.
29. Total number of anchor rods per abutment (no units) – If applicable, enter the total number of anchor rods per abutment. This input value must be a whole number between 1 and 16.
30. Anchor rod diameter (in.) – If applicable, enter the anchor rod diameter, 30, as shown in Figure 3.3a.
31. Height of anchor block (ft) – If applicable, enter the height of the anchor block, 31, as shown in Figure 3.3a.
32. Bottom elevation of anchor block (ft) – If applicable, enter the vertical distance between the stream elevation and bottom of the anchor block, 32, as shown in Figure 3.3a. This input

value is limited such that the bottom and top anchor block faces must be between the stream and roadway elevations, respectively.

33. Anchor rod length for this analysis (ft) – If applicable, enter the anchor rod length,(33), as shown in Figure 3.3a. This value must be greater than or equal to the minimum anchor rod length provided in the cell directly above this input cell. This minimum value is determined by the FDT and ensures that the buried concrete anchor block is beyond the passive and active soil failure planes as shown in Figure 2.3.

Once the required input values have been entered in the highlighted cells, and if no red text warning messages appear, the adequacy of the pile system can be verified. This is accomplished by clicking the 'Check Pile Design' button located below the last input cell as shown in Figure 3.4. The engineer must click this button each time changes are made to any of the input values previously designated.

3.2.1.3. DESIGN CHECKS

The next section of the PDW displays the various design requirements for steel piles in a cohesive or cohesionless soil. A brief explanation of the various strength and serviceability requirements is also presented. Additionally, suggestions for adjusting the previously described input values to satisfy these design requirements are also included in this section. As shown in Figure 3.5, each design requirement is assigned a number that corresponds to the description provided in this section.

1. Axial pile stress (ksi) – The total axial pile stress must be less than the allowable stress limits cited in Section 6.2.6.1 of the Iowa DOT BDM. If this requirement is not satisfied, the engineer could:
   - Increase the number of piles (input Cell 4).
   - Use a pile with a larger cross sectional area (input Cell 18 or 19).
   - Use a less conservative (i.e., calculate a more accurate value) dead load and/or live load abutment reaction (input Cells 8 and 9, respectively).
2. Pile bearing capacity (kips) – The total axial pile load must be less than the bearing capacity. The bearing capacity of a friction pile will be sufficient if the embedded length is greater than or equal to the minimum length specified in the Foundation Summary section (Cell 13) of the PDW (shown in Figure 3.5 and discussed later in this chapter). If this requirement is not satisfied for end bearing and combination end and friction bearing piles, the engineer could:

Design Checks					
	1	Axial pile stress	$\frac{P}{A} \le \sigma_{ALL}$	5.83 ksi	OK
	2	Pile bearing capacity	Axial Pile Load ≤ Capacity	727.6 kip	OK
	3	Interaction equation validation	$\frac{1}{(1 - f_a/F'_e)} > 1.0$	1.04	OK
	4	Combined loading interaction requirement # 1	$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{(1 - f_a/F'_{ex})F_{bx}} \le 1.0$	0.43	OK
	5	Combined loading interaction requirement # 2	$\frac{f_a}{0.60F_y} + \frac{f_{bx}}{F_{bx}} \le 1.0$	0.60	OK
	6	Buried anchor block location	Anchor rod length ≥ minimum	16.00 ft	OK
	7	Anchor rod stress	$\sigma \le 0.6F_y$	16.1 ksi	OK
	8	Anchor block capacity	Total Anchor Force ≤ Capacity	10.8 kip per pile	OK
	9	Maximum displacement	$\delta_{MAX} \le 1.5$ in.	0.21 in.	OK

Anchor Design Worksheet (if applicable)

Foundation Summary			
	1	Roadway width	24.00 ft
	2	Span length	60.00 ft
	3	Distance between superstructure bearings and roadway grade	3.00 ft
	4	Backwall height	8.00 ft
	5	Dead load abutment reaction	180.9 kip per abutment
	6	Live load abutment reaction	121.5 kip per abutment
	7	Number of piles	6
	8	Total axial pile load	36.1 tons
	9	Pile spacing	4.50 ft
	10	Pile size	HP10x42
	11	Pile yield stress	36 ksi
	12	Minimum total pile length	47 ft

Figure 3.5. Design Checks and Foundation Summary section of the FDT PDW for steel piles.

- Increase the number of piles (input Cell 4).
- If applicable, use an alternative pile size that provides a larger friction bearing resistance per foot (input Cells 13, 14, and 18 through 25).
- If applicable, use an alternative pile size with a larger end bearing area (input Cell 18 or 19).
- Use a less conservative (i.e., calculate a more accurate value) dead load and/or live load abutment reaction (input Cells 8 and 9, respectively).

3. <u>Interaction equation validation (non-dimensional)</u> – The secondary pile moment factor must be greater than or equal to one. If this requirement is not satisfied, the engineer could:
   - Increase the number of piles (input Cell 4).
   - Use an alternative pile size with a larger axial capacity (input Cell 18 or 19 through 25).
   - Use an alternative lateral restraint system or configuration (input Cells 27 through 33).
   - Use a pile with a higher yield stress (input Cell 17).
   - Use a less conservative (i.e., calculate a more accurate value) dead load and/or live load abutment reaction (input Cells 8 and 9, respectively).
4. <u>Combined loading interaction requirement # 1 (non-dimensional)</u> – This is the first of two AISC Manual interaction equations. As previously noted, similar interaction equations are also provided in AASHTO. This equation (Equation E.1 of Appendix E) must yield a value less than or equal to one. If this requirement is not satisfied, the engineer could:
   - Increase the number of piles (input Cell 4).
   - Use an alternative pile size with a larger axial and flexural capacity (input Cell 18 or 19 through 25).
   - Use an alternative lateral restraint system or configuration (input Cells 27 through 33).
   - Use a pile with a higher yield stress (input Cell 17).
   - Use a less conservative (i.e., calculate a more accurate value) dead load and/or live load abutment reaction (input Cells 8 and 9, respectively).
5. <u>Combined loading interaction requirement # 2 (non-dimensional)</u> – This is the second of two AISC Manual interaction equations (Equation E.5 of Appendix E). Again, similar interaction equations are also provided in AASHTO. This interaction equation must yield a value less than or equal to one. If this requirement is not satisfied, the engineer could use the recommendations provided for the previous pile interaction requirement (design check Cell 5).
6. <u>Buried anchor block location (ft)</u> – The length of the anchor rod must be long enough to ensure the failure planes of the anchor block and backwall do not intersect as shown in Figure 2.3. If this requirement is not satisfied, the engineer could:
   - Increase the anchor rod length (input Cell 33).

- Adjust the distance between the bottom face of the anchor block and the stream elevation (input Cell 32).

7. Anchor rod stress (ksi) – The applied anchor rod stress must be less than or equal to 60 percent the yield stress as specified by the AISC Manual. If this requirement is not satisfied, the engineer could:
   - Increase the number of anchor rods per abutment (input Cell 29).
   - Increase the anchor rod diameter (input Cell 30).
   - Use an anchor rod with a higher yield stress (input Cell 28).
   - Increase the number of piles to reduce the required anchor rod force (input Cell 4).
   - Use an alternative pile size with an increased flexural capacity to reduce the required anchor rod force (input cell 18 or 19 through 25).
8. Anchor block capacity (kip per pile) – The lateral anchor force per pile must be less than the maximum passive resistance of the soil surrounding the anchor block. The maximum lateral capacity per pile and computed anchor force per pile are provided directly below input Cell 32 as shown in Figure 3.4. The anchor capacity per pile is based on the soil pressure distribution of Figure E.1 and Equation E.14 in Appendix E. The computed anchor force per pile is determined by the FDT using indeterminate structural analysis as described in Chapter 2. If this requirement is not satisfied, the engineer could:
   - Increase the height of the anchor block (input Cell 31).
   - Decrease the distance between the bottom face of the anchor block and the stream elevation (input Cell 32).
   - Use an alternative pile size with a larger flexural capacity to reduce the required anchor force per pile (input Cell 18 or 19 through 25).
9. Maximum displacement (in.) – AASHTO, Section 4 defines in the maximum allowable horizontal substructure displacement as 1.5 in. If this requirement is not satisfied, the engineer could:
   - Increase the number of piles (input Cell 4).
   - Use an alternative pile size with a larger flexural rigidity (input Cell 18 or 19 through 25).
   - Use an alternative lateral restraint system or configuration (input Cells 27 through 33).

3.2.1.4. INFORMATION SUMMARY

As shown in Figure 3.5 the PDW also contains a Foundation Summary section. Each summary cell is assigned a number that corresponds to the description provided in this section. Items 1, 2, 4 through 7, and 11 are provided by the engineer.

1. Roadway width (ft)
2. Span length (ft)
3. Distance between superstructure bearings and roadway grade (ft) – This cell contains the combined depth of the superstructure plus roadway as determined by the FDT.
4. Backwall height (ft)
5. Dead load abutment reaction (kips per abutment)
6. Live load abutment reaction (kips per abutment)
7. Number of piles (no units)
8. Total axial pile load (tons) – This cell contains the total axial pile load as determined by the FDT. This value includes the sum of the dead and live load axial pile loads (both multiplied by the nominal axial pile factor as described in Chapter 2) and the pile self-weight.
9. Pile spacing (ft) – This cell contains the pile spacing as determined by the FDT.
10. Pile size (no units) – This cell contains the standard pile shape for this analysis as indicated by the engineer. If a non-standard pile shape size was used, this summary cell indicates a reference to the pile property input cells.
11. Pile yield stress (ksi)
12. Minimum total pile length (ft) – This cell contains the minimum total pile length as determined by the FDT. For end bearing and combination bearing piles, the minimum total pile length is equal the vertical distance between the superstructure bearings and the location of adequate end bearing material. For friction bearing piles, the minimum required pile length is equal to the vertical distance between the stream elevation and the superstructure bearings plus the depth required for adequate bearing capacity.
13. Minimum embedded pile length (ft) – If the pile is designed as a friction pile, this cell contains the minimum required embedded pile length for friction pile as determined by the FDT.

3.2.2. Timber Piles in a Cohesive or Cohesionless Soil

3.2.2.1. INSTRUCTIONS WORKSHEET

The IW provides a brief description of the input quantities required for the PDW. A portion of the IW for timber piles is shown in Figure 3.6. Also, the IW contains a figure of an abutment cross section and roadway cross section near the abutment which is reproduced in Figure 3.7. This figure provides a graphical representation of some of the required input parameters. Each circled number in Figure 3.7 corresponds to an input cell number on the IW and PDW for timber piles (Figures 3.6 and 3.8, respectively). Once the IW has been reviewed, the engineer may proceed by clicking the 'Pile Design Worksheet' button (in the upper left corner as shown in Figure 3.6).

3.2.2.2. REQUIRED INPUT

This section provides a detailed explanation of the input values required for the PDW for a timber pile. As shown in Figure 3.8, each input cell is highlighted. The quantities shown in the highlighted input cells of Figure 3.8 are not applicable for all bridge sites and are shown for demonstration purposes only. The only difference between the PDW for timber piles in a cohesive or cohesionless soil is the required soil input parameter (undrained shear strength and soil friction angle, respectively).

1. Span length (ft) – Enter the bridge span length as measured from the centerlines of the bridge abutments. This input value is limited to a value between 20 and 90 ft.
2. Roadway width (ft) – Enter the bridge roadway width. This input value must be greater than or equal to 24 ft.
3. Location of the exterior pile relative to the edge of the roadway (ft) – Enter the horizontal distance, 3, between the centerline of the exterior pile and the roadway edge as shown in Figure 3.7b. This value, limited to plus or minus 5 ft, is positive if all piles are located within the exterior limits of the roadway as shown in Figure 3.7b.
4. Number of piles (no units) – Enter the number of piles. This value must be a whole number that falls within the range specified in the two cells located directly above this input cell. The range of piles provided is based on the roadway width, location of the exterior pile relative to the edge of the roadway (input Cells 2 and 3, respectively), and spacing limitations cited in section 6.2.4 of the Iowa DOT BDM.
5. Backwall height (ft) – Enter the vertical distance, 5, between the stream elevation and roadway elevation as shown in Figure 3.7a.

County:
Project No:
Description:

computed by:
checked by:
date:

The calculations performed in the Pile Design Worksheet are based on the guidelines of the AASHTO Standard Specifications, the AISC Manual of Steel Construction, the Iowa DOT Bridge Design Manual (Iowa DOT BDM), and the National Design Specifications Manual for Wood Construction (NDS Manual).

Once the instructions on this worksheet have been reviewed, proceed to the Pile Design Worksheet or return to the pile and soil selection worksheet by clicking the icons below.

Pile Design Worksheet

Return to Pile and Soil Selection Worksheet

Data required is to be entered in the highlighted cells of the Pile Design Worksheet.

The following numbers and explanations correspond the highlighted cells on the Pile Design Worksheet; all circled numbers are shown on the figure provided.

Cell No.	Description
1	Enter the span length between the centerline of the abutment bearings.
2	Enter the roadway width of the bridge.
3	Enter the distance between the centerline of the exterior pile and the edge of the roadway. This value is positive for situations when the exterior pile is within the limits of the roadway width as shown above.
4	Enter the number of piles per abutment. This value must be within the range given in the cells directly above this input cell.
5	Enter the vertical distance between the roadway grade and the stream elevation.
6	Enter the vertical distance from the stream elevation to the estimated depth of scour. This value is based on stream hydraulics, geological information, and engineering judgment.
7	Use the pull-down menu provided to select the superstructure system for this analysis
8	Enter the dead load abutment reaction for this analysis. A default value maybe provided in the cell directly above this input cell.
9	Enter the live load abutment reaction for this analysis. A default value is provided in the cell directly above this input cell.
10	Enter the average standard penetration test (SPT) blow count (N-value) for the upper level soil.
11	Enter the undrained shear strength of the soil for this analysis. A default value base of the SPT N-value is provided in the cell located directly above this input cell.
12	Enter the friction bearing resistance per foot of pile for soils *within* 30 ft of the natural ground line. Appendix C of Volume I provides friction bearing values based on the SPT N-value.
13	Enter the friction bearing resistance per foot of pile for soils *not within* 30 ft of the natural ground line. Appendix C of Volume I provides friction bearing values based on the SPT N-value.
14	Use the pull-down menu provided to select the timber species for this analysis.
15	Enter the tabulated timber bending stress.
16	Enter the tabulated timber compressive stress (parallel to the grain).
17	Enter the tabulated modulus of elasticity.
18	Enter the pile butt diameter (i.e., the driving end).
19	Enter the pile tip diameter (i.e., the embedded end).
20	Enter the vertical distance between the stream elevation and the superstructure bearing points.
21	Use the pull-down menu provided to select the type of lateral restraint system (if any) for this analysis.
22	If applicable, use the pull-down menu provided to select the anchor rod yield stress.
23	If applicable, enter the total number of anchor rods for one abutment.
24	If applicable, enter the anchor rod diameter.

Figure 3.6. Selected portion of the FDT IW for timber piles.

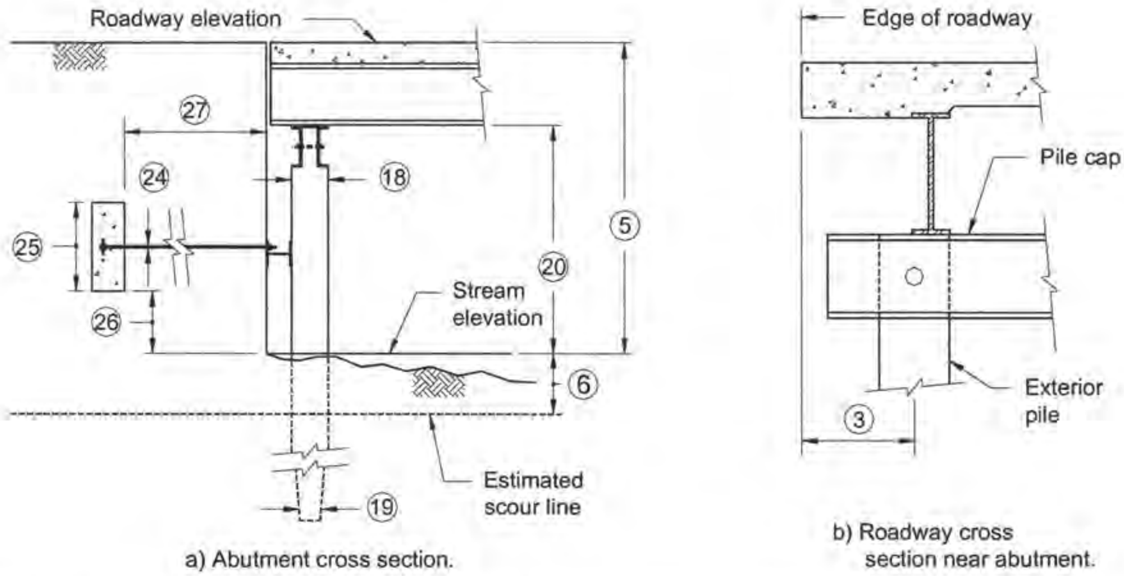


Figure 3.7. Graphical representation of various input requirements for timber piles.

6. Estimated scour depth (ft) – Enter the estimated depth of soil, 6, that could potentially be eroded away due to scour as shown in Figure 3.7a. This value should be based on hydraulic and geological information as well as engineering judgment.
7. Superstructure system (no units) – Use the pull-down menu provided to select the appropriate superstructure being used.
8. Dead load abutment reaction for this analysis (kips per abutment) – Enter the dead load abutment reaction for this analysis. If a 24 or 30 ft roadway width and a superstructure system other than a BISB and RRFC are used, a conservative value will be shown in the cell located directly above this input cell as shown in Figure 3.8. This default value is based on span length, roadway width, and the superstructure used (input Cells 1, 2, and 7, respectively).
9. Live load abutment reaction for this analysis (kips per abutment) – Enter the live load abutment reaction for this analysis. A conservative value is provided in the cell directly above this input cell as shown in Figure 3.8. This default value is based on the span length and roadway width (input Cells 1 and 2, respectively).
10. Soil SPT blow count (N) – Enter the SPT blow count for the soil in the immediate vicinity of the foundation piles. If a non-uniform soil profile is present, use the average blow count for the upper level soil. This input value must be a whole number between 1 and 50.

County:
Project No:
Description:

computed by:
checked by:
date:

Instructions Worksheet

Return to Pile and Soil Selection Worksheet

General Bridge Input	1	Span length	40.00	ft
	2	Roadway width	24.00	ft
	3	Location of exterior pile relative to the edge of the roadway	0.92	ft
		Maximum number of piles	9	piles on 2.77 ft centers
		Minimum number of piles	4	piles on 7.39 ft centers
	4	Number of piles	8	
	5	Backwall height	6.00	ft
	6	Estimated scour depth	2.00	ft
	7	Superstructure system	PCDT	
		Estimated dead load abutment reaction	128.6	kip per abutment (default value)
	8	Dead load abutment reaction for this analysis	128.6	kip per abutment
		Estimated live load abutment reaction	110.0	kip per abutment (default value)
	9	Live load abutment reaction for this analysis	110.0	kip per abutment
Foundation Material Input	10	Soil SPT blow count (N)	20	
		Correlated soil friction angle ( )	33.3	degrees
	11	Soil friction angle for this analysis	33.3	degrees
	12	Estimated friction bearing value for depths less than 30 ft	0.7	tons per ft
	13	Estimated friction bearing value for depths greater than 30 ft	0.7	tons per ft
Pile Input	14	Timber species	southern pine	
	15	Tabulated timber bending stress	1,750	psi
	16	Tabulated timber compressive stress	1,100	psi
	17	Tabulated timber modulus of elasticity	1,600,000	psi
	18	Pile butt diameter	13.0	in.
	19	Pile tip diameter	10.0	in.
Lateral Restraint Input	20	Superstructure bearing elevation	3.58	ft
	21	Type of backwall lateral restraint	buried concrete anchor block	
	22	Anchor rod yield stress	60	ksi
	23	Total number of anchor rods per abutment	5	per abutment
	24	Anchor rod diameter	0.75	in.
	25	Height of anchor block	3.00	ft
	26	Bottom elevation of anchor block	1.08	ft
		Anchor block geotechnical capacity	8.3	kip per pile
		Computed anchor force per pile	6.3	kip per pile
		Minimum anchor rod length	13.47	ft
	27	Anchor rod length for this analysis	15.00	ft

Check Pile Design

Figure 3.8. Input section of the FDT PDW for timber piles.

11. Soil undrained shear strength for this analysis, **for timber piles in a cohesive soil only** (psf)

    Enter the undrained shear strength (c_U); a default value based on the most commonly used correlation of the SPT blow count and undrained shear strength as reported by Terzaghi and Peck [11] is provided in the cell directly above this input cell. This input cell is not shown in Figure 3.8 in lieu of the soil friction angle. This relationship is provided as Equation 3.1. Since this correlation can be unreliable for some in-situ conditions, it is recommended that the undrained shear strength be determined by testing soil samples from the bridge site. This input value is used to calculate the depth of pile fixity for piles in cohesive soils, the equation for which is presented in Appendix E.

11. Soil friction angle for this analysis, **for timber piles in a cohesionless soil only** (degrees) Enter the soil friction angle (ϕ); a default value, based on the correlation of the SPT blow count and the soil friction angle as reported by Peck [12] is provided in the cell directly above this input cell as shown in Figure 3.8. This relationship is provided as Equation 3.2. It is recommended that the soil friction angle be determined from tests on soil samples from the bridge site. This input value is used to calculate the depth of pile fixity for piles in cohesionless soils, the equation for which is presented in Appendix E.
12. Estimated friction bearing value for depths less than 30 ft (tons per ft) – Enter an estimated friction bearing resistance for the soil *within* 30 ft of the natural ground line. Estimated values for this input parameter can be obtained from Appendix B or the Iowa DOT FSIC. This input value must be between 0.1 and 2.0 tons per foot.
13. Estimated friction bearing value for depths greater than 30 ft (tons per ft) – Enter an estimated friction bearing resistance for soils *not within* 30 ft of the natural ground line. Estimated values for this input parameter can be obtained from Appendix B or the Iowa DOT FSIC. This input value must be between 0.1 and 2.0 tons per foot.
14. Timber species (no units) – Use the provided pull-down menu to select the timber species for this analysis.
15. Tabulated timber bending stress (psi) – Enter the tabulated timber bending stress. AASHTO Table 13.5.1A. recommends a tabulated timber bending stress of 1,750 psi for both southern pine and douglas fir timber species (structural grade lumber).
16. Tabulated timber compressive stress (psi) – Enter the tabulated timber compressive stress (parallel to the grain). AASHTO Table 13.5.1A. recommends tabulated compressive stress values of 1,100 and 1,350 psi for southern pine and douglas fir timber species, respectively (structural grade lumber).
17. Tabulated timber modulus of elasticity (psi) – Enter the tabulated timber modulus of elasticity. AASHTO Table 13.5.1A. recommends tabulated timber modulus of elasticity values of 1,600,000 and 1,700,000 psi for southern pine and douglas fir timber species, respectively (structural grade lumber).

18. Pile butt diameter (in.) – Enter the diameter of the pile as measured at the butt or pile driving end, 18, as shown in Figure 3.7a. This input value must be greater than or equal to 10 in. as required by the Iowa DOT Standard Specifications [13].
19. Pile tip diameter (in.) – Enter the diameter of the pile as measured at the tip or embedded end, 19, as shown in Figure 3.7a. This input value must be greater than or equal to 6 in. as required by the Iowa DOT Standard Specifications.
20. Superstructure bearing elevation (ft) – Enter the vertical distance between the stream elevation and superstructure bearings, 20, as shown in Figure 3.7a. This input value must be between 0 ft and the backwall height (input Cell 5).
21. Type of backwall lateral restraint system (no units) – Use the provided pull-down menu to select the lateral restraint system for this analysis.
22. Anchor rod yield stress (ksi) – If applicable, use the pull down menu provided to select the anchor rod yield stress.
23. Total number of anchor rods per abutment (no units) – If applicable, enter the total number of anchor rods per abutment. This input value must be a whole number between 1 and 16.
24. Anchor rod diameter (in.) – If applicable, enter the anchor rod diameter, 24, as shown in Figure 3.7a.
25. Height of anchor block (ft) – If applicable, enter the height of the anchor block, 25, as shown in Figure 3.7a.
26. Bottom elevation of anchor block (ft) – If applicable, enter the vertical distance between the stream elevation and bottom of the anchor block, 26, as shown in Figure 3.7a. This input value is limited such that the bottom and top anchor block faces must be between the stream and roadway elevations, respectively.
27. Anchor rod length for this analysis (ft) – If applicable, enter the anchor rod length, 27, as shown in Figure 3.7a. This value must be greater than or equal to the minimum anchor rod length provided in the cell directly above this input cell. This minimum value is determined by the FDT and ensures that the buried concrete anchor block is beyond the passive and active soil failure planes as shown in Figure 2.3.

Once the required input values have been entered in the highlighted cells, and if no red text warning messages appear, the adequacy of the pile system can be verified. This is accomplished by clicking the 'Check Pile Design' button located below the last input cell as shown in Figure 3.8. The engineer must click this button each time changes are made to any of the input values previously designated.

3.2.2.3. DESIGN CHECKS

The next section of the PDW displays the various design requirements for timber piles in a cohesive or cohesionless soil. A brief explanation of the various strength and serviceability requirements is also presented. Additionally, suggestions for adjusting the previously described input values to satisfy these design requirements are also included in this section. As shown in Figure 3.9, each design requirement is assigned a number that corresponds to the description provided in this section.

1. Axial pile load (kips) – The total axial pile load for a timber pile must be less than the allowable limit cited in Section 6.2.6.1 of the Iowa DOT BDM. The maximum axial load for a timber pile with a length between 20 and 30 ft is 20 tons. However, this allowable load can be increased to 25 tons per pile if the pile length is greater than 30 ft. If this requirement is not satisfied, the engineer could:
   - Increase the number of piles (input Cell 4).
   - Use a less conservative (i.e., calculate a more accurate value) dead load and/or live load abutment reaction (input Cells 8 and 9, respectively).
2. Pile length(ft) – The length of a timber pile must be between 20 and 55 ft as cited by Section 6.2.6.1 of the Iowa DOT BDM. If this requirement is not satisfied, the engineer could:
   - Increase the number of piles (input Cell 4).
   - Use a larger diameter pile to increase the friction bearing resistance per foot of pile thus reducing the required pile length (input cells 18 and 19).
   - Use a less conservative (i.e., calculate a more accurate value) dead load and/or live load abutment reaction (input Cells 8 and 9, respectively).
3. Pile bearing capacity (kips) – The total axial pile load must be less than the bearing capacity. The bearing capacity of a friction pile will be sufficient if the embedded length is greater than or equal to the minimum length provided for this design requirement as shown in Figure 3.9.
4. Interaction equation validation (non-dimensional) – The secondary pile moment factor must be less than or equal to one. If this requirement is not satisfied, the engineer could:
   - Increase the number of piles (input Cell 4).

Design Checks	1	Axial pile load	$\boxed{P \le P_{ALLOWABLE}}$	42.1 kip	OK
	2	Pile length	$\boxed{\text{Length} \le 55 \text{ ft}}$	33 ft	OK
	3	Pile bearing capacity	$\boxed{\text{Axial Pile Load} \le \text{Capacity}}$	sufficient if embedded pile length is at least	30 ft
	4	Interaction equation validation	$\boxed{\frac{1}{(1-f_C/F'_e)} > 1.0}$	1.04	OK
	5	Combined loading interaction requirement $\boxed{\left(\frac{f_C}{F'_C}\right)^2 + \frac{f_{bx}}{F'_b\left(1-\frac{f_C}{F'_{ex}}\right)} + \frac{f_{by}}{F'_b\left(1-\frac{f_C}{F'_{ey}}-\left(\frac{f_{bx}}{F'_{bE}}\right)^2\right)} \le 1.0}$		0.61	OK
	6	Buried anchor block location	$\boxed{\text{Anchor rod length} \ge \text{minimum}}$	15.00 ksi	OK
	7	Anchor rod stress	$\boxed{\sigma \le 0.6F_y}$	22.9 ksi	OK
	8	Anchor block capacity	$\boxed{\text{Total Anchor Force} \le \text{Capacity}}$	8.3 kip per pile	OK
	9	Maximum displacement	$\boxed{\delta_{MAX} \le 1.5 \text{ in.}}$	0.35 in.	OK

Anchor Design Worksheet (if applicable)

Foundation Summary	1	Roadway width	24.00 ft
	2	Span length	40.00 ft
	3	Distance between superstructure bearings and roadway grade	2.42 ft
	4	Backwall height	6.00 ft
	5	Dead load abutment reaction	128.6 kip per abutment
	6	Live load abutment reaction	110.0 kip per abutment
	7	Number of piles	8
	8	Total axial pile load	21.0 tons
	9	Pile spacing	3.17 ft
	10	Pile size	
		Butt diameter	13.0 in.
		Tip diameter	10.0 in.
	11	Pile material properties	
		Timber species	southern pine
		Tabulated timber compressive stress	1,100 psi
		Tabulated timber bending stress	1,750 psi
		Tabulated timber modulus of elasticity	1,600,000 psi
	12	Minimum total pile length	33 ft

Figure 3.9. Design Checks and Foundation Summary sections of the FDT PDW for timber piles.

- Use a larger pile diameter to increase the axial capacity (input Cells 18 and 19).
- Use an alternative lateral restraint system or configuration (input Cells 21 through 27).
- Use a timber species with a higher tabulated compressive stress (input Cells 14 and 16).
- Use a less conservative (i.e., calculate a more accurate value) dead load and/or live load abutment reaction (input Cells 8 and 9, respectively).

5. Combined loading interaction requirement (non-dimensional) – The NDS Manual interaction equation (Equation E.7 in Appendix E) must yield a value less than or equal to one. If this requirement is not satisfied, the engineer could:
   - Increase the number of piles (input Cell 4).
   - Use a larger pile diameter which increases the axial and flexural capacity of the pile (input Cells 18 and 19).
   - Use a timber species with a higher tabulated timber bending and axial stress (input Cells 14 through 16).
   - Use an alternate lateral restraint system or configuration (input Cells 21 through 27).
   - Use a less conservative (i.e., calculate a more accurate value) dead load and/or live load abutment reaction (input Cells 8 and 9, respectively).
6. Buried anchor block location (ft) – The length of the anchor rod must be long enough to ensure the failure planes of the anchor block and backwall do not intersect as shown in Figure 2.3. If this requirement is not satisfied, the engineer could:
   - Increase the anchor rod length (input Cell 27).
   - Adjust the distance between the bottom face of the anchor block and the stream elevation (input Cell 26).
7. Anchor rod stress (ksi) – The applied anchor rod stress must be less than 60 percent the yield stress as specified by the AISC Manual. If this requirement is not satisfied, the engineer could:
   - Increase the number of anchor rods per abutment (input Cell 23).
   - Increase the diameter of the anchor rods (input Cell 24).
   - Use an anchor rod with a higher yield stress (input Cell 22).
   - Use a larger pile diameter which increases the flexural capacity and reduces the required anchor rod force (input Cells 18 and 19).
   - Increase the number of piles to reduce the required anchor rod force (input Cell 4).
8. Anchor block capacity (kips per pile) – The lateral anchor force per pile must be less than the maximum passive resistance of the soil surrounding the anchor block. The maximum lateral capacity per pile and computed anchor force per pile are provided below input Cell 26 as shown in Figure 3.8. The anchor capacity per pile is based on the soil pressure distribution of Figure E.1 and Equation E.14 in Appendix E. The computed anchor force

per pile is determined by the FDT using indeterminate structural analysis as described in Chapter 2. If this requirement is not satisfied, the engineer could

- Increase the height of the anchor block (input Cell 25).
- Decrease the distance between the bottom face of the anchor and the stream elevation (input Cell 26).
- Use a larger diameter pile which will increase the pile flexural capacity and reduce the required anchor force per pile (input Cells 18 and 19).

9. <u>Maximum displacement (in.)</u> – AASHTO, Section 4 defines in the maximum allowable horizontal substructure displacement as 1.5 in. If this requirement is not satisfied, the engineer could:
   - Increase the number of piles (input Cell 4).
   - Use a larger diameter pile which increases the flexural rigidity of the pile (input Cells 18 and 19).
   - Use an alternative lateral restraint system or configuration (input Cells 21 through 27).

3.2.2.4. INFORMATION SUMMARY

As shown in Figure 3.9, the PDW also contains a Foundation Summary section. Each summary value is assigned a number that corresponds to the description provided in this section. Items 1, 2, 4 through 7, 10, and 11 are provided by the engineer.

1. <u>Roadway width (ft)</u>
2. <u>Span length (ft)</u>
3. <u>Distance between superstructure bearings and roadway grade (ft)</u> – This cell contains the combined depth of the superstructure plus roadway as determined by the FDT.
4. <u>Backwall height (ft)</u>
5. <u>Dead load abutment reaction (kip per abutment)</u>
6. <u>Live load abutment reaction (kip per abutment)</u>
7. <u>Number of piles (no units)</u>
8. <u>Total axial pile load (tons)</u> – This cell contains the total axial pile load as determined by the FDT. This value includes the sum of the dead and live load axial pile loads (both multiplied by the nominal axial pile factor as described in Chapter 2), and the pile self-weight.
9. <u>Pile spacing (ft)</u> – This cell contains the pile spacing as determined by the FDT.

10. Pile size (in.) – These cells provide the pile butt and tip diameters.
11. Pile material properties (psi) – These cells provide the timber species, tabulated compressive stress, tabulated bending stress, and elastic modulus.
12. Minimum total pile length (ft) – This cell contains the minimum total pile length required as determined by the FDT. The minimum required pile length is equal to the vertical distance between the stream elevation and the superstructure bearings plus the depth required for friction bearing capacity.

3.2.3. Anchor Design Worksheet

The Anchor Design Worksheet (ADW) is only required if the buried concrete anchor block option is selected in the PDW (input Cells 27 and 21 for steel and timber piles, respectively). The ADW provided is applicable to all combinations of piles and soil types. If applicable, the engineer may proceed by clicking the 'ADW (if applicable)' button shown in Figures 3.5 and 3.9 (for steel and timber piles, respectively) once all the design requirements have been satisfied in the PDW. In the ADW, additional input information such as the anchor material properties and reinforcement details are provided by the engineer. This input information, which is briefly described in the Instructions section of the ADW, is used to calculate the internal anchor block forces, determine the structural capacity, and check a series of design requirements. Other information required for the design of the reinforced concrete anchor block (e.g., the anchor height and anchor rod force) are entered in, or determined by the PDW. A summary of the anchor system details is also provided in the ADW.

3.2.3.1. INSTRUCTIONS

The Instructions section of the ADW provides a brief description of the input required as shown in Figure 3.10. Additionally, the Instructions section also contains a figure of an anchor cross section and plan view of the reinforced concrete anchor block which is reproduced as Figure 3.11. This figure provides a graphical representation of some anchor block quantities required from the engineer. Each circled number in Figure 3.11 corresponds to cell number in the Instructions and Input section of the ADW (Figures 3.10 and 3.12, respectively). The height of the reinforced concrete anchor block, denoted as 'b' in Figure E.1 of Appendix E, is not a required input value for the ADW. This PDW input value is shown as 31 and 25 in Figures 3.3 and 3.7, respectively. The width of the anchor block, which is used to calculate the effective depth of the concrete, is set at 12 in. for this analysis.

County:
Project No:
Description:

computed by:
checked by:
date:

THIS WORKSHEET IS ONLY TO BE USED AFTER THE PILE SYSTEM HAS BEEN DESIGNED AND ALL DESIGN REQUIREMENTS HAVE BEEN SATISFIED.

Return to Pile Design Worksheet

Go to Pile and Soil Selection Worksheet

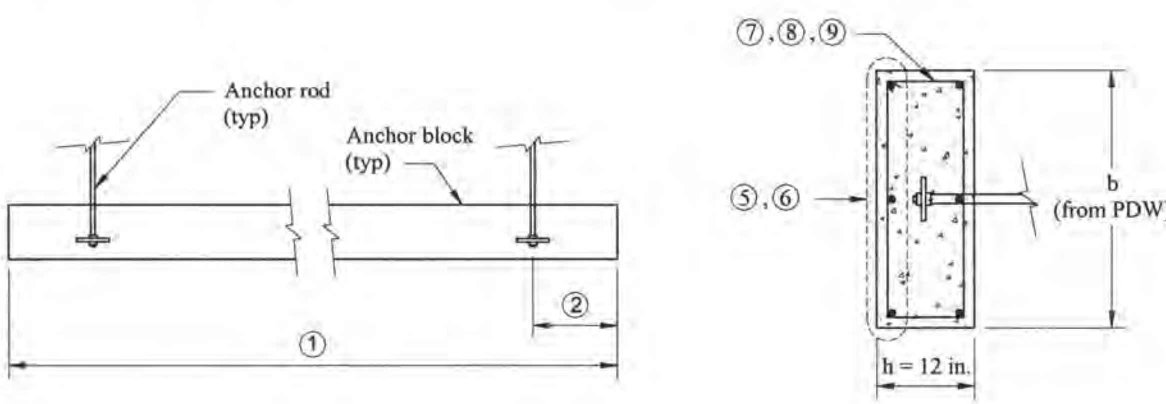
The design in this worksheet is based on Section 8 of the AASHTO Standard Specifications.

Once the instructions on this sheet have been reviewed, proceed to the input section of this worksheet below.

Data required is to be entered in the highlighted cells of the Input Information section; all cirlced numbers are shown on the figure provided.

Instructions	Cell No.	Description
	1	Enter the total length of the anchor block.
	2	Enter the distance between the end of the anchor block and the exterior anchor rod.
	3	Enter the anchor block concrete compressive strength.
	4	Use the pull-down menu provided to select the yield strength of the reinforcing steel.
	5	Enter the number of tension steel reinforcing bars on one vertical anchor block face.
	6	Use the pull-down menu provided to select the tension steel bar size.
	7	If applicable, use the pull-down menu provided to select the stirrup bar size.
	8	If applicable, enter the number of stirrup legs per section.
	9	If applicable, enter the stirrup spacing for this analysis. This value must be less than the value in the cell directly above this input cell.

Figure 3.10. Selected portion of the FDT ADW Instructions.



a) Anchor block plan view.

b) Anchor block cross section.

Figure 3.11. Graphical representation of selected input values for the ADW.

3.2.3.2. REQUIRED INPUT

A brief explanation of the input information required from the engineer for the ADW, shown in Figure 3.12, is presented in this section. Each input cell is highlighted and assigned a number that corresponds to the description provided in this section. The quantities shown in the highlighted input cells of Figure 3.12 are shown for clarity and are not applicable for all bridge sites.

1. Anchor block length (ft) – Enter the total length of the anchor block, 1, as shown in Figure 3.11a. This input value must be greater than or equal to the product of the pile spacing and number of piles which accounts for an additional one-half pile space for each exterior pile.
2. Distance from the end of the anchor block to exterior anchor rod (ft) – Enter the distance between the end of the anchor block and the exterior anchor rod, 2, as shown in Figure 3.11a. This input value must be greater than or equal to 1 ft.
3. Concrete compressive strength (ksi) – Enter the compressive strength of the concrete to be used in the anchor block. As a minimum, 3 ksi was selected for this input value, however a higher concrete compressive strength can be entered.
4. Yield strength of reinforcing steel (ksi) – Use the provided pull-down menu to select the reinforcement yield stress. Once the first four input quantities previously described have been entered, click the '$(A_S)_{REQ'D}$' button as shown in Figure 3.12 to determine the area of steel required for strength. The required tension steel area is determined by reinforced concrete design equations, the anchor block dimensions, material properties entered, and the maximum factored moment as determined by the FDT using the moment distribution method and AASHTO load combinations.
5. Number of tension steel bars on one vertical anchor block face (no units) – Enter the number of tension steel bars located on one vertical anchor block face, 5, shown in Figure 3.11b. This input cell, in addition to input Cell 6, determines the tension steel area provided for one vertical anchor block face. The provided tension steel area must be greater than the required tension steel area for strength that is given directly above this input cell as shown in Figure 3.12. This input value must be a whole number that, when evenly spaced, provides a spacing of less than 18 in. as required by Section 8.21.6 of AASHTO.

Input Information	1	Anchor block length		28.00 ft		
	2	Distance from end of anchor block to exterior anchor rod		2.00 ft		
	3	Concrete compressive strength		3.0 ksi		
	4	Yield strength of reinforcing steel		60 ksi		
		Determine tension steel area required for strength	$(A_S)_{REQ'D}$			
		Tension steel area required		0.24 in.^{2}		
	5	Number of tension steel bars on one vertical anchor block face		3 bars		
	6	Tension steel bar size		5 #		
		Minimum tension steel area		0.92 in.^{2}		
		Are stirrups required?		Yes		
	7	Shear stirrup bar size number		3 #		
	8	Number of stirrup legs per section		2		
		Maximum stirrup spacing		4.66 in.		
	9	Stirrup spacing for this analysis		4.50 in.		

Design Checks	1	Design flexural capacity	$\boxed{M_U < \phi M_N}$	37.32 ft-kips	OK	{AASHTO 8.16.3.2}
	2	Reinforcement ratio	$\boxed{\rho < 0.75\rho_b}$	0.0027	OK	{AASHTO 8.16.3.2.2}
	3	Minimum reinforcement			OK	{AASHTO 8.17}
	4	Design shear capacity	$\boxed{V_U < \phi V_N}$	54.5 kip	OK	{AASHTO 8.16.6.1.1}

Anchor System Summary	1	Number of anchor rods		5		
	2	Anchor rod yield stress		60 ksi		
	3	Anchor rod diameter		0.750 in.		
	4	Anchor rod length		15.00 ft		
	5	Anchor rod spacing		6.00 ft		
	6	Vertical distance between bottom of anchor block and roadway grade		4.92 ft		
	7	Anchor block length		28.00 ft		
	8	Anchor block height		3.0 ft		
	9	Anchor block width		12.0 in.		
	10	Concrete compressive strength		3.0 ksi		
	11	Details for reinforcement on vertical anchor block face		3	#	5 bars
	12	Details for shear stirrups	#	3	bars on	4.50 in. centers

Figure 3.12. Input Information, Design Checks, and Anchor System Summary sections of the FDT ADW.

6. Tension steel bar size (no units) – Use the provided pull-down menu to select the tension steel bar size, 6, as shown in Figure 3.11b. As previously discussed, this input cell in addition to input Cell 5, is used to determine the area of steel provided which must be greater than the required steel area provided directly above input Cell 5 as shown in Figure 3.12.

7. Shear stirrup bar size (no units) – If shear stirrups are required as indicated by cell located directly above this input cell (shown in Figure 3.12), use the pull-down menu provided to select the shear stirrup bar size,(7). A shear stirrup is identified in Figure 3.11b.
8. Number of stirrup legs per section (no units) – If shear stirrups are required, enter the number of stirrup legs per section,(8). This input value must be a whole number that is greater than or equal to one. A shear stirrup is identified in Figure 3.11b.
9. Stirrup spacing for this analysis (in.) – If shear stirrups are required, enter the stirrup center-to-center spacing,(9). This input value must be less than the value provided directly above this input cell as shown in Figure 3.11b. The minimum stirrup spacing is the minimum of: 1.) the maximum spacing allowed to obtain the necessary design shear capacity, 2.) the maximum spacing allowed if stirrups are not required for strength, 3.) one-half the effective depth of the concrete, and 4.) 24 in.

3.2.3.3. DESIGN CHECKS

The next section of the ADW displays the various design requirements for the reinforced concrete anchor block. A brief explanation of the structural and serviceability requirements follows. Additionally, suggestions for adjusting the previously described input values to satisfy these design requirements are also included in this section. As shown in Figure 3.12, each design requirement is assigned a number that corresponds to the description provided in this section.

1. Design flexural capacity (ft-kips) – The maximum factored bending moment, which is determined by the ADW using the moment distribution method and AASHTO load combinations, must be less than the design flexural capacity of the anchor block as specified by AASHTO, Section 8. If this requirement is not satisfied, the engineer could:
   - Redesign the anchor block section.
   - Use an alternate pile and anchor rod configuration to possibly reduce the required anchor rod force and corresponding internal anchor block bending loads (input cells located in the PDW).
2. Reinforcement ratio (non-dimensional) – The reinforcement ratio of the anchor block must be less than 75 percent of the reinforcement ratio associated with a balanced condition, both of which are defined in AASHTO, Section 8. If this requirement is not satisfied, the engineer could:
   - Increase the width of the concrete compression block by increasing the height of the anchor block (input cell located in the PDW).

- Increase the concrete compressive strength (input Cell 3).
- Redesign the anchor block section.

3. Minimum reinforcement (no units) – The cracking moment, multiplied by a factor of 1.2, must be less than the design flexural capacity of the anchor block. Alternatively, this requirement can be waived if the area of tension steel provided (input Cells 5 and 6) is at least four-thirds the minimum steel area required. If this requirement is not satisfied, the engineer could:
   - Decrease the compressive strength of the concrete (input Cell 4).
   - Use a smaller anchor block height to reduce the gross moment of inertia (input cell in the PDW).
   - Increase the design flexural capacity of the anchor block as previously described.
4. Design shear capacity (kips) – The maximum factored shear force must be less than the design shear capacity of the anchor block as specified by AASHTO, Section 8. The design shear capacity is the sum of the concrete shear strength and the additional capacity provided by shear stirrups. If this requirement is not satisfied, the engineer could:
   - Increase the compressive strength of the concrete (input Cell 3).
   - Decrease the shear stirrup spacing (input Cell 9).
   - Use a larger shear stirrup bar size (input Cell 7).
   - Increase the number of stirrup legs per section (input Cell 8).
   - Increase the height of the anchor block thus increasing the concrete shear strength (input cell in the PDW).

3.2.3.4. INFORMATION SUMMARY

As shown in Figure 3.12, the ADW also contains an Anchor System Summary section. Each summary value has been assigned a number that corresponds to the brief description that follows. Note that quantities 1 through 4, 7, 8, and 10 through 12 have been entered by the engineer.

1. Number of anchor rods (no units)
2. Anchor rod yield stress (ksi)
3. Anchor rod diameter (in.)
4. Anchor rod length (ft)
5. Anchor rod spacing (ft) – This cell contains the anchor rod spacing as determined by the FDT.

6. Vertical distance between bottom of anchor block and roadway grade (ft) – This cell contains the vertical distance between the bottom of the anchor block and roadway elevation as determined by the FDT.
7. Anchor block length (ft)
8. Anchor block height (ft)
9. Anchor block width (in.) – This cell contains the width of the concrete anchor block which is set to 12 in. for all designs in the ADW.
10. Concrete compressive strength (ksi)
11. Details for reinforcement on vertical anchor block face (various units) – This cell contains the tensile reinforcement details. This includes the number of tension steel bars on each vertical anchor block face in addition to the bar size.
12. Details for shear stirrups (various units) – If applicable, this cell contains the shear stirrup reinforcement details. This includes the bar size in addition to the shear stirrup spacing.

3.3. STANDARD ABUTMENT PLANS

An example of the generic standard abutment plans that were developed for this project is presented in Appendix D. CAD computer files of the complete standard plan set are provided in the TR-486 final report. The CAD computer files will produce full size (11 in. by 17 in.) sheets. Additionally, the full size sheets can be easily modified to produce larger construction sheets.

The standard abutment plans can be used by Iowa County Engineers to produce the necessary drawings for the more common LVR bridge abutments systems. Using the various superstructures and the associated standard plans previously developed by the BEC, the engineer can generate a complete set of bridge plans. It should be noted that by modifying the bearing surface of the standard abutment systems provided, essentially any type of bridge superstructure system can be supported.

In order for the engineer to produce a finished set of abutment plans, the necessary details such as the bridge geometry, member size designations (i.e., W, C, and HP shapes), and material properties must be inserted in the spaces provided. The pre-designed foundations systems, which are discussed later in this chapter, or the FDT provide many of the necessary details for the generic standard abutment plans.

As shown in Appendix D, the standard abutment plans provided consist of two different types of sheets. The first type consists of two general sheets that will be used for all bridge abutments and are both included in the final set of construction sheets. These include the cover sheet (Sheet 1) and a general bridge plan and elevation layout sheet (Sheet 2). The second type of sheets consist of a series of construction sheets (Sheets 3a, 3b, etc.) with different combinations of pile caps, backwall systems, anchor systems, and pile types. At most, two of these construction sheets will be required for a particular bridge site (i.e., a different construction sheet for each bridge abutment). If the two bridge abutments use the same combination of previously mentioned substructure variables, the same sheet can be used twice with different dimensions, if necessary.

4. VERIFICATION OF THE FOUDATION DESIGN TEMPLATE

As previously stated, complete design examples which demonstrate the application of the design methodology and the foundation design template were completed but are not included herein. These include the use of the FDT for two foundation systems. These calculations not only demonstrate the application of the design methodology developed for this project but can also be used to verify the accuracy of the FDT. The following list provides a general description of the full design examples completed.

Example 1: In the first set of calculations demonstrates the design methodology for determining the foundation loads, performing the structural analysis, and calculating the capacity of timber piles with an anchor system. In this example, an abutment is designed for a PCDT superstructure with a span length and roadway width of 40 and 24 ft, respectively. The timber piles are embedded in a soil that is best described in the Iowa DOT FSIC as a gravelly sand with an average SPT blow count of 21. The backwall height and estimated depth of scour are equal to six and two feet, respectively.

Example 2: The second set of calculations demonstrates a design methodology for determining the foundation loads, performing the structural analysis, and calculating capacity for steel piles without an anchor system. In this example, an abutment is designed for a PSC superstructure with a span length and roadway width of 60 and 30 ft, respectively. The steel piles are embedded in soil that is best described in the Iowa DOT FSIC as a firm, glacial clay with a SPT blow count of 11. The backwall height and estimated depth of scour are equal to eight and two feet, respectively.

Several computer models were also developed using structural analysis software for the previously described lateral substructure loadings to verify the internal system forces and deflections computed by the FDT. These computer models consisted of both determinate (i.e., without an anchor) and indeterminate (i.e., with an anchor) systems. Additionally, computer models were developed to verify the internal pile forces and deflections computed by the FDT if a positive connection between the superstructure and substructure is used.

5. USERS MANUAL SUMMARY

This research project consisted of three major phases: the collection of information for LVR bridge abutments, the development of an easy-to-use design methodology, and the creation of several substructure design aids for the Iowa County Engineers. In the first phase, a literature review and survey of the Iowa County Engineers was completed. The literature review focused on locating LVR bridge abutment information and standard abutment plans. A survey of the Iowa counties was conducted to determine the use of standard abutment plans by the counties and the identification of common construction methods and trends. In this phase of the project, several LVR bridge abutment systems commonly used by the Iowa counties, a series of possible alternative abutment systems, and two different pile analysis methodologies that could be used to investigate the influence of the lateral and vertical loadings on the piles were identified.

The second phase of this project involved investigating different analysis methodologies and the development of a design methodology for the different foundation elements. Two lateral load analysis methods were investigated including a linear and non-linear method. It was found that each method has certain advantages such as the ability to model complex soil conditions and profiles, accurately representing the actual soil and pile interaction, and the ease of incorporating the analysis method into a complete design methodology. It was decided that the linear analysis procedure presented by Broms [6, 7] would be the most suitable for this project based on the relative simplicity and correlation of the calculated maximum pile moment when compared to the non-linear analysis method. This method considers the pile fixed at a calculated depth below ground level based on soil and pile properties in addition to lateral loading conditions. A design methodology used to determine the structural capacity of the steel and timber piles was developed using the recommendations of AASHTO, the AISC Manual, and the NDS Manual.

An analysis and design methodology was also developed for a lateral restraint system that can potentially be used to resist the lateral substructure loads. Two lateral restraint systems are presented including a positive connection between the superstructure and substructure, and a buried anchor block connected to the substructure with the use of anchor rods. If a positive connection is used, the longitudinal stiffness of the superstructure is assumed to transfer lateral loads between the substructure units. The lateral restraint provided by an anchor system is a result of the passive soil pressure that acts on the vertical anchor block face. This passive soil resistance force is transferred to the substructure through anchor rods and an abutment wale. A procedure for determining the structural capacity of the anchor block was developed using the reinforced concrete design specifications in AASHTO.

The third and final phase of this project involved the development of LVR bridge abutment design aids. These design aids include the FDT and a series of generic standard abutment plans. The FDT is used to verify the adequacy of a pile and anchor system for a particular bridge and site. Information such as the bridge geometry, soil conditions, pile information, and lateral restraint details are provided by the engineer. This information is used to determine the substructure loads, perform a structural analysis of the foundation elements, determine the respective capacities, and perform a series of design checks. The various generic standard abutment plans include general information and instruction sheets in addition to construction sheets with different combinations of substructure details.

6. AKNOWLEDGEMENTS

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Mark J. Nahra: County Engineer, Delaware County

Tom P. Schoellen: Assistant County Engineer, Black Hawk County

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7. REFERENCES

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Additional related references are provided in Volume I of this report.

APPENDIX A

ESTIMATED GRAVITY LOADS

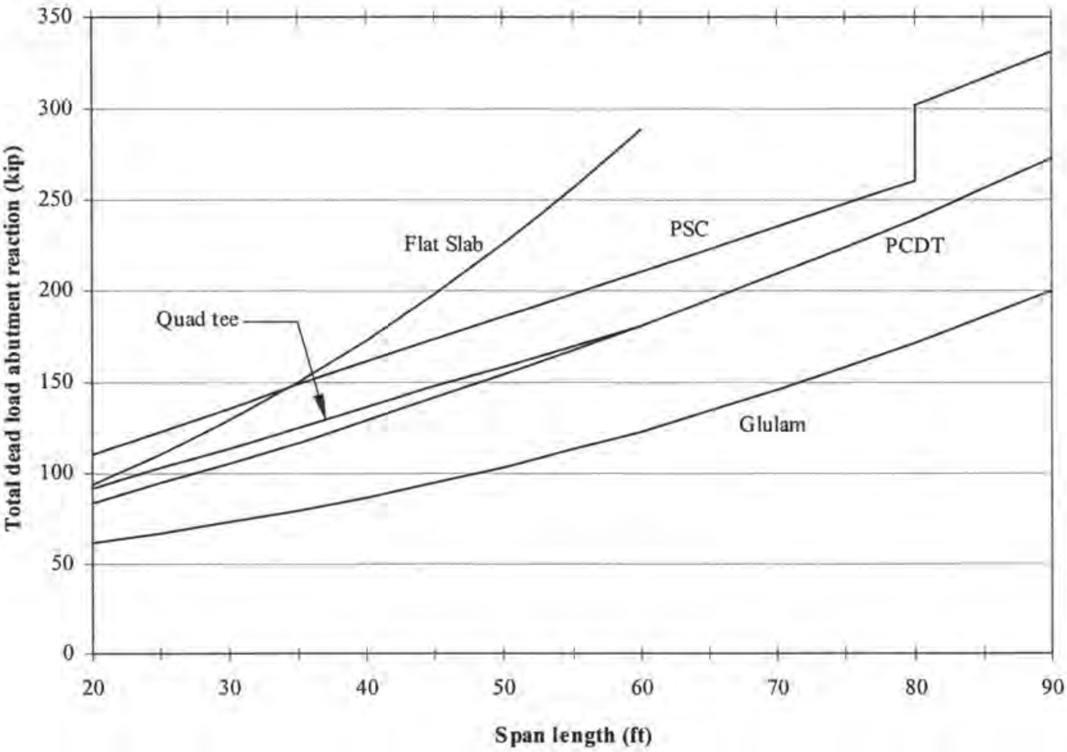


Figure A.1. Estimated dead load abutment reactions for a 24 ft roadway width.

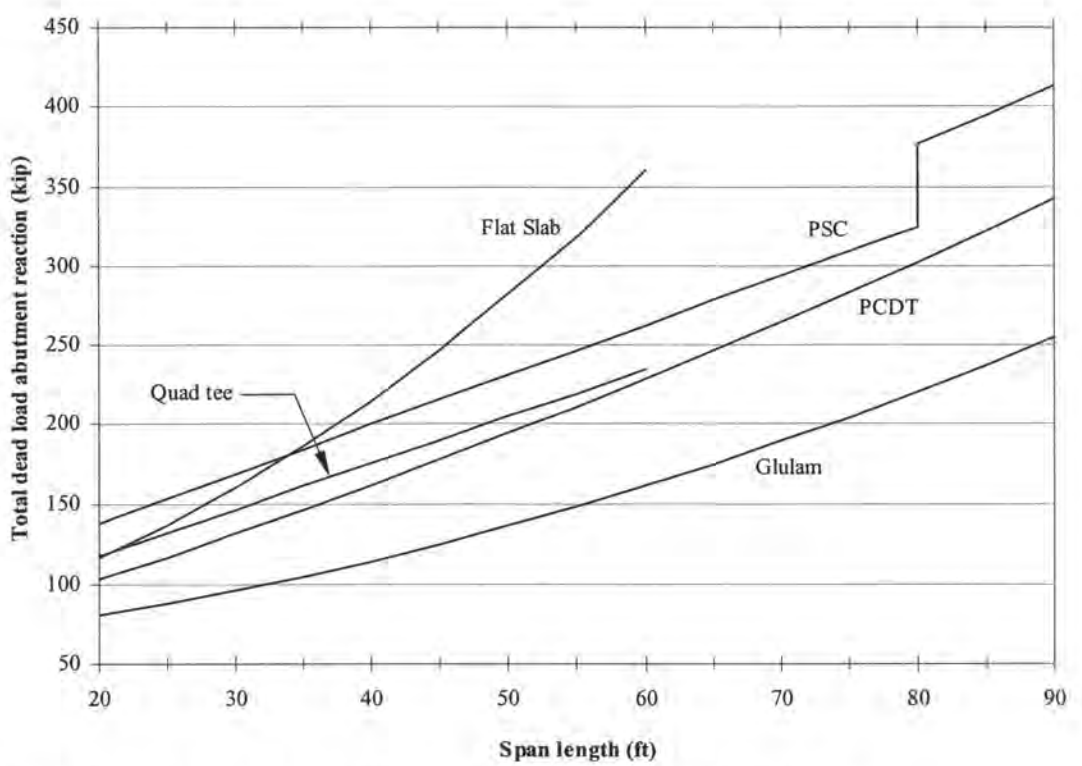


Figure A.2. Estimated dead load abutment reactions for a 30 ft roadway width.

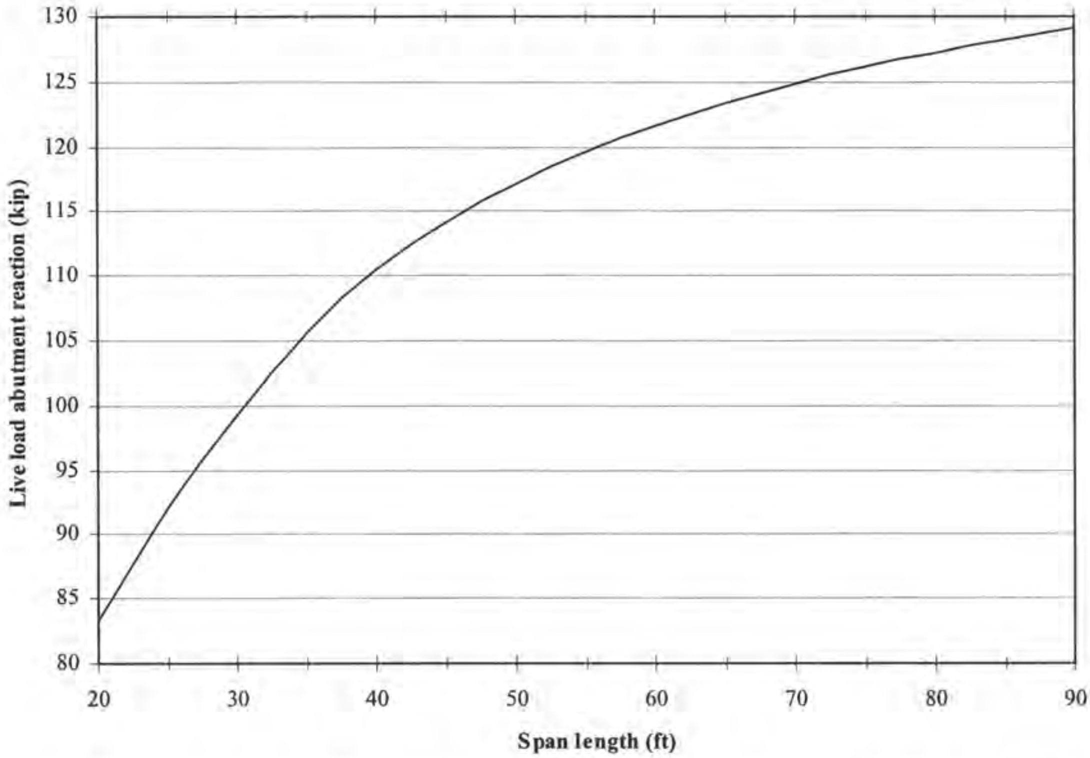


Figure A.3. Estimated live load abutment reactions without impact for two 10 ft design traffic lanes.

APPENDIX B

DRIVEN PILE FOUNDATION SOILS INFORMATION CHART

Table B.1. Estimated end bearing values for steel H-piles.

Soil Description	SPT Blow Count (N) Mean	Range	* Estimated End Bearing Values for Steel H-Piles (psi)
Granular Material	-	0 - 25	0
	-	25 - 50	1,000 - 2,000
	-	50 - 100	2,000 - 4,000
	-	100 - 300	4,000 - 8,000
Bedrock	-	> 300	9,000
	-	100 - 200	6,000
	-	> 200	9,000
Cohesive Material	12	10 - 50	0
	20	-	500
	25	-	1,000
	50	-	2,000
	100	-	3,500

* End bearing values include a factor of safety equal to 2.0.

NOTE: Table B.1 is adapted from the Iowa DOT Foundation Soils Information Chart (1994), Table 1.1

Table B.2. Estimated friction bearing values for steel H-piles and 10 in. diameter timber piles.

Soil Description	SPT Blow Count (N)		* Estimated Friction Bearing Values (tons / ft)			
			Timber	Steel H-Piles		
	Mean	Range	** 10 in.	HP 10	HP 12	HP 14
Alluvium or Loess						
Very Soft Silty Clay	1	0 - 1	0.2	0.1	0.2	0.2
Soft Silty Clay	3	2 - 4	0.3	0.2	0.3	0.3
Stiff Silty Clay	6	4 - 8	0.4	0.3	0.4	0.5
Firm Silty Clay	11	7 - 15	0.6	0.5	0.6	0.7
Stiff Silt	6	3 - 7	0.4	0.3	0.4	0.4
Stiff Sandy Silt	6	4 - 8	0.4	0.3	0.4	0.4
Stiff Sandy Clay	6	4 - 8	0.4	0.3	0.4	0.5
Silty Sand	8	3 - 13	0.3	0.3	0.3	0.4
Clayey Sand	13	6 - 20	0.5	0.4	0.5	0.7
Fine Sand	15	8 - 22	0.6	0.5	0.6	0.7
Coarse Sand	20	12 - 28	0.8	0.7	0.8	0.9
Gravelly Sand	21	11 - 31	0.8	0.7	0.8	0.9
Granular Material	> 40	-	1.0	1.0	1.2	1.4
Glacial Clay						
Firm Silty Glacial Clay	11	7 - 15	0.7	0.6	0.7	0.8
Firm Clay (Gumbotil)	12	9 - 15	0.7	0.6	0.7	0.8
Firm Glacial Clay (depths > 30 ft)	11	7 - 15	0.6 (0.8)	0.7 (0.8)	0.8 (1.0)	0.9 (1.1)
Firm Sandy Glacial Clay (depths > 30 ft)	13	9 - 15	0.6 (0.8)	0.7 (0.8)	0.8 (1.0)	0.9 (1.1)
Firm - Very Firm Glacial Clay (depths > 30 ft)	14	11 - 17	0.7 (0.9)	0.7 (1.0)	0.8 (1.2)	0.9 (1.4)
Very Firm Glacial Clay (depths > 30 ft)	24	17 - 30	0.7 (0.9)	0.7 (1.0)	0.8 (1.2)	0.9 (1.4)
Very Firm Sandy Glacial Clay (depths > 30 ft)	25	15 - 30	0.8 (1.0)	0.7 (1.0)	0.8 (1.2)	0.9 (1.4)
Cohesive of Glacial Material (depths > 30 ft)	> 35	-	0.8 (1.0)	0.7 (1.0)	0.8 (1.2)	0.9 (1.4)

* Friction bearing values include a factor of safety equal to 2.0.

** Friction bearing values for other than a 10 in. diameter pile = chart value * pile diameter / 10.

NOTE: Table B.2 is adapted from the Iowa DOT Foundation Soils Information Chart (1994), Table 1.1.

APPENDIX C

PRINTOUTS FROM THE FOUNDATION DESIGN TEMPLATE

START UP WORKSHEET

County:
Project No:
Description:

computed by:
checked by:
date: 7/21/2004

Please select the pile type and soil type for this analysis by clicking the corresponding button below.

Steel Piles In A Cohesive Soil	Steel Piles In A Cohesionless Soil	Timber Piles In A Cohesive Soil	Timber Piles In A Cohesionless Soil

Although all design checks are completed by this spreadsheet, however the developer cannot be held responsible.

STEEL PILES IN A COHESIVE SOIL
INSTRUCTIONS AND PILE DESIGN WORKSHEET

County:
Project No:
Description:

computed by:
checked by:
date: 7/21/2004

THIS WORKSHEET IS ONLY FOR STEEL PILES IN A COHESIVE SOIL.

The calculations performed in the Pile Design Worksheet are based on the guidelines of the AASHTO Standard Specifications, the AISC Manual of Steel Construction, and the Iowa DOT Bridge Design Manual (Iowa DOT BDM).

Once the instructions in this worksheet have been reviewed, proceed to the Pile Design Worksheet or return to the pile and soil selection worksheet by clicking the icons below.

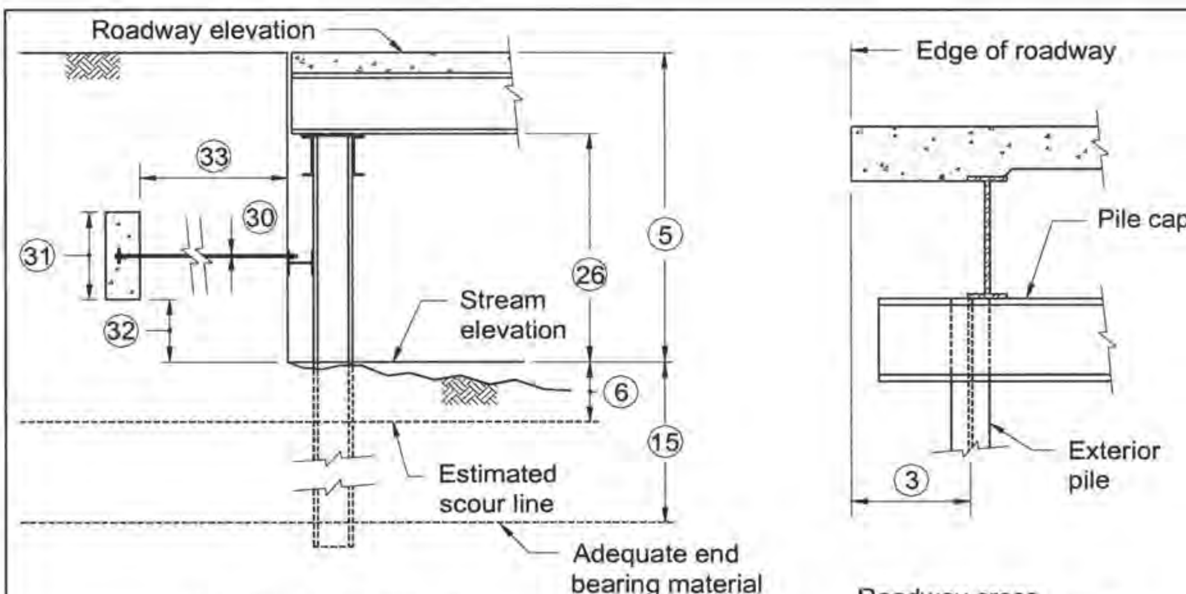
Pile Design Worksheet

Return to Pile and Soil Selection Worksheet

Data required is to be entered in the highlighted cells of the Pile Design Worksheet.

The figure below is to be used as a reference for the various input dimensions.

The stream elevation is the datum for all elevations.



Abutment cross section.

Roadway cross section near abutment.

County:
Project No:
Description:

computed by:
checked by:
date: 7/21/2004

THIS WORKSHEET IS ONLY FOR STEEL PILES IN A COHESIVE SOIL.

The following numbers and explanations correspond the highlighted cells on the Pile Design Worksheet; all circled numbers are shown on the figure above.

Cell No.	Description
1	Enter the span length between the centerline of the abutment bearings.
2	Enter the roadway width of the bridge.
3	Enter the distance between the centerline of the exterior pile and the edge of the roadway. This value is positive for situations when the exterior pile is within the limits of the roadway width as shown above.
4	Enter the number of piles per abutment. This value must be within the range given in the cells directly above this input cell.
5	Enter the vertical distance between the roadway grade and the stream elevation.
6	Enter the vertical distance from the stream elevation to the estimated depth of scour. This value is based on stream hydraulics, geological information, and engineering judgment.
7	Use the pull-down menu provided to select the type of superstructure system for this analysis.
8	Enter the dead load abutment reaction for this analysis. A default value may be provided in the cell directly above this input cell.
9	Enter the live load abutment reaction for this analysis. A default value is provided directly above this input cell.
10	Enter the average standard penetration test (SPT) blow count (N-value) for the upper level soil.
11	Enter the undrained shear strength of the soil for this analysis. A default value based on the SPT N-value is provided in the cell directly above this input cell.
12	Use the pull-down menu provided to select the type of pile bearing resistance for gravity loads. NOTE: End bearing is only allowed in bed rock for this spreadsheet.
13	If applicable, enter the friction bearing resistance per foot of pile, for the soil *within* 30 ft of the natural ground line. Appendix C of Volume I provides friction bearing values based on the SPT N-value.
14	If applicable, enter the friction bearing resistance per foot of pile, for the soil *not within* 30 ft of the natural ground line. Appendix C of Volume I provides friction bearing values based on the SPT N-value.
15	If applicable, enter the estimated depth to adequate end bearing foundation material.
16	If applicable, use the pull-down menu provided to select the SPT N-value for the end bearing foundation material.
17	Use the pull-down menu provided to select the pile yield stress.
18	Use the pull-down menu provided to select an H-pile shape. If a standard shape is selected, input values for cell 19 through 25 will not be required from the engineer.
19	If applicable, enter the cross-sectional area of the pile.
20	If applicable, enter the pile width measured parallel to the backwall.
21	If applicable, enter the strong axis moment of inertia (the strong axis is assumed parallel to the backwall).
22	If applicable, enter the strong axis section modulus (the strong axis is assumed parallel to the backwall).
23	If applicable, enter the weak axis section modulus (the weak axis is assumed perpendicular to the backwall).

County:
Project No:
Description:

computed by:
checked by:
date: 7/21/2004

BRIDGE ENGINEERING CENTER

THIS WORKSHEET IS ONLY FOR STEEL PILES IN A COHESIVE SOIL.

24	If applicable, enter the strong axis radius of gyration (the strong axis is assumed parallel to the backwall.)
25	If applicable, enter the weak axis radius of gyration (the weak axis is assumed perpendicular to the backwall.)
26	Enter the vertical distance between the stream elevation and the superstructure bearing points.
27	Use the pull-down menu provided to select the type of lateral restraint system (if any) for this analysis.
28	If applicable, use the pull-down menu provided to select the anchor rod yield stress.
29	If applicable, enter the total number of anchor rods for one abutment.
30	If applicable, enter the anchor rod diameter.
31	If applicable, enter the height of the anchor block .
32	If applicable, enter the vertical distance between the stream elevation and the bottom of the anchor block.
33	If applicable, enter the anchor rod length for this analysis. This value must be greater than or equal to the value given directly above this input cell.

County:
Project No:
Description:

computed by:
checked by:
date: 7/21/2004

THIS WORKSHEET IS ONLY FOR STEEL PILES IN A COHESIVE SOIL.

Instructions Worksheet

Go to Pile and Soil Selection Worksheet

General Bridge Input	1	Span length	60.00	ft		
	2	Roadway width	24.00	ft		
	3	Location of exterior pile relative to the edge of the roadway	0.50	ft		
		Maximum number of piles	10	piles on	2.56	ft centers
		Minimum number of piles	4	piles on	7.67	ft centers
	4	Number of piles	6			
	5	Backwall height	6.00	ft		
	6	Estimated scour depth	2.00	ft		
	7	Superstructure system	PCDT			
		Estimated dead load abutment reaction	180.9	kip per abutment (default value)		
	8	Dead load abutment reaction for this analysis	180.9	kip per abutment		
		Estimated live load abutment reaction	121.5	kip per abutment (default value)		
	9	Live load abutment reaction for this analysis	121.5	kip per abutment		
Foundation Material Input	10	Soil SPT blow count (N)	10			
		Correlated soil un-drained shear strength (C_u)	1,270	psf		
	11	Soil undrained shear strength for this analysis	1,270	psf		
	12	Type of vertical pile bearing resistance	friction & end bearing			
	13	Estimated friction bearing value for depths less than 30 ft	0.7	tons per ft		
	14	Estimated friction bearing value for depths greater than 30 ft	0.8	tons per ft		
	15	Depth to adequate end bearing foundation material	40	ft		
	16	SPT blow count for end bearing foundation material	100 < N < 200			
Pile Input	17	Pile yield stress	36	ksi		
	18	Select pile type	HP10x42			
	19	Pile cross sectional area	12.4	sq. in.		
	20	Pile flange width	10.1	in.		
	21	Pile moment of inertia (strong axis)	210	in.^4		
	22	Pile section modulus (strong axis)	43.4	cubic in.		
	23	Pile section modulus (weak axis)	14.2	cubic in.		
	24	Pile radius of gyration (strong axis)	4.13	in.		
	25	Pile radius of gyration (weak axis)	2.41	in.		
Lateral Restraint Input	26	Superstructure bearing elevation	3.50	ft		
	27	Type lateral restraint system	buried concrete anchor block			
	28	Anchor rod yield stress	60	ksi		
	29	Total number of anchor rods per abutment	5	per abutment		
	30	Anchor rod diameter	0.75	in.		
	31	Height of anchor block	2.50	ft		
	32	Bottom elevation of anchor block	2.00	ft		
		Anchor block geotechnical capacity	8.1	kip per pile		
		Computed anchor force per pile	6.4	kip per pile		
		Minimum anchor rod length	12.15	ft		
	33	Anchor rod length for this analysis	14.00	ft		

County:
Project No:
Description:

BRIDGE ENGINEERING CENTER

computed by:
checked by:
date: 7/21/2004

THIS WORKSHEET IS ONLY FOR STEEL PILES IN A COHESIVE SOIL.

Check Pile Design

Geotechnical, Structural and Serviceability Requirements

Design Checks	1	Axial pile stress	$P/A \le \sigma_{ALL}$	5.84 ksi	OK
	2	Pile bearing capacity	Axial Pile Load ≤ Capacity	111.6 kip	OK
	3	Interaction equation validation	$\frac{1}{(1 - f_a/F'_e)} > 1.0$	1.03	OK
	4	Combined loading interaction requirement # 1	$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{(1 - f_a/F'_{ex}) F_{bx}} \le 1.0$	0.43	OK
	5	Combined loading interaction requirement # 2	$\frac{f_a}{0.60F_y} + \frac{f_{bx}}{F_{bx}} \le 1.0$	0.60	OK
	6	Anchor rod stress	$\sigma \le 0.6F_y$	17.5 ksi	OK
	7	Anchor block capacity	Total Anchor Force ≤ Capacity	8.1 kip per pile	OK
	8	Maximum displacement	$\delta_{MAX} \le 1.5$ in.	0.171 in.	OK

Anchor Design Worksheet (if applicable)

Foundation Summary	1	Roadway width	24.00 ft
	2	Span length	60.00 ft
	3	Distance between superstructure bearings and roadway grade	2.50 ft
	4	Backwall height	6.00 ft
	5	Dead load abutment reaction	180.9 kip per abutment
	6	Live load abutment reaction	121.5 kip per abutment
	7	Number of piles	6
	8	Total axial pile load	36.2 tons
	9	Pile spacing	4.60 ft
	10	Pile size	HP10x42
	11	Pile yield stress	36 ksi
	12	Minimum total pile length	44 ft

STEEL PILES IN COHESIONLESS SOIL
INSTRUCTIONS AND PILE DESIGN WORKSHEET

County:
Project No:
Description:

computed by:
checked by:
date: 7/21/2004

THIS WORKSHEET IS ONLY FOR STEEL PILES IN A COHESIONLESS SOIL.

The calculations performed in the Pile Design Worksheet are based on the guidelines of the AASHTO Standard Specifications, the AISC Manual of Steel Construction, and the Iowa DOT Bridge Design Manual (BDM).

Once the instructions in this worksheet have been reviewed, proceed to the Pile Design Worksheet or return to the pile and soil selection worksheet by clicking the icons below.

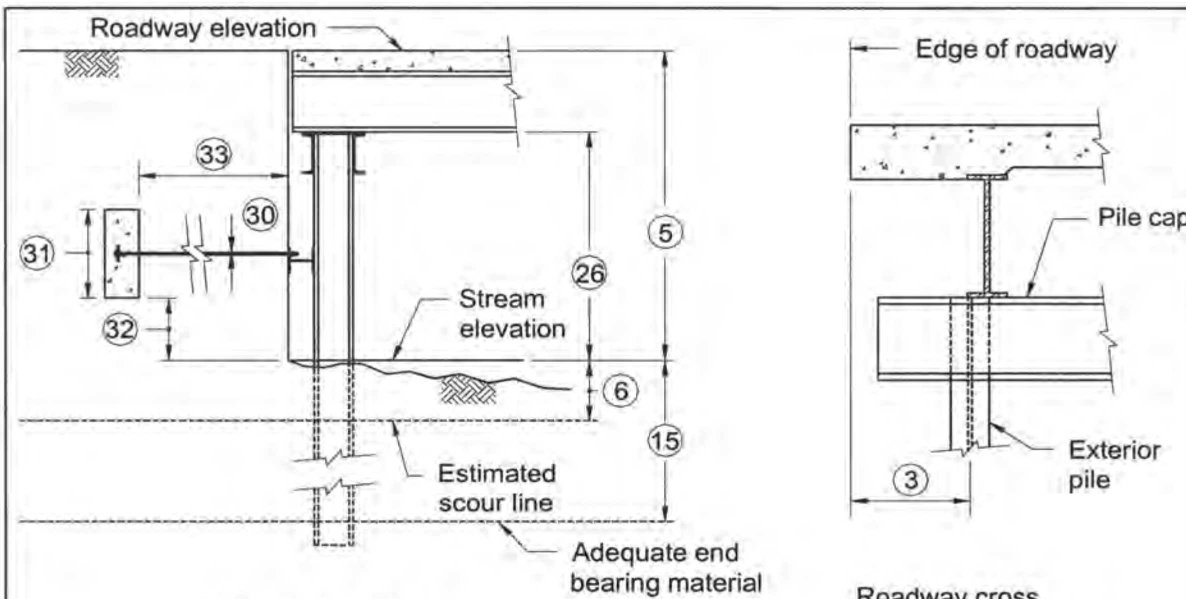
Pile Design Worksheet

Return to Pile and Soil Selection Worksheet

Data required is to be entered in the highlighted cells of the Pile Design Worksheet.

The stream elevation is the datum for all elevations.

The figure below is to be used as a reference for the various input dimensions.



Abutment cross section.

Roadway cross section near abutment.

County:
Project No:
Description:

computed by:
checked by:
date: 7/21/2004

THIS WORKSHEET IS ONLY FOR STEEL PILES IN A COHESIONLESS SOIL.

The following numbers and explanations correspond the highlighted cells on the Pile Design Worksheet; all circled numbers are shown on the figure above.

Cell No.	Description
1	Enter the span length between the centerline of the abutment bearings.
2	Enter the roadway width of the bridge.
3	Enter the distance between the centerline of the exterior pile and the edge of the roadway. This value is positive for situations when the exterior pile is within the limits of the roadway width as shown above.
4	Enter the number of piles per abutment. This value must be within the range given in the cells directly above this input cell.
5	Enter the vertical distance between the roadway grade and the stream elevation.
6	Enter the vertical distance from the stream elevation to the estimated depth of scour. This value is based on stream hydraulics, geological information, and engineering judgment.
7	Use the pull-down menu provided to select the type of superstructure system for this analysis.
8	Enter the dead load abutment reaction for this analysis. A default value may be provided in the cell directly above this input cell.
9	Enter the live load abutment reaction for this analysis. A default value is provided directly above this input cell.
10	Enter the average standard penetration test (SPT) blow count (N-value) for the upper level soil.
11	The soil friction angle to be used for this analysis. The engineer can either use the default value given directly above this input cell or a value based of bridge site soil tests.
12	Use the pull-down menu provided to select the type of pile bearing resistance for gravity loads. NOTE: End bearing is only allowed in bed rock for this spreadsheet.
13	If applicable, enter the friction bearing resistance per foot of pile, for the soil *within* 30 ft of the natural ground line. Appendix C of Volume I provides friction bearing values based on the SPT N-value.
14	If applicable, enter the friction bearing resistance per foot of pile, for the soil *not within* 30 ft of the natural ground line. Appendix C of Volume I provides friction bearing values based on the SPT N-value.
15	If applicable, enter the estimated depth to adequate end bearing foundation material.
16	If applicable, use the pull-down menu provided to select the SPT N-value for the end bearing foundation material.
17	Use the pull-down menu provided to select the pile yield stress.
18	Use the pull-down menu provided to select an H-pile shape. If a standard shape is selected, input values for cell 19 through 25 will not be required from the engineer.
19	If applicable, enter the cross-sectional area of the pile.
20	If applicable, enter the pile width measured parallel to the backwall.
21	If applicable, enter the strong axis moment of inertia (the strong axis is assumed parallel to the backwall).
22	If applicable, enter the strong axis section modulus (the strong axis is assumed parallel to the backwall).
23	If applicable, enter the weak axis section modulus (the weak axis is assumed perpendicular to the backwall).

County:
Project No:
Description:

computed by:
checked by:
date: 7/21/2004

THIS WORKSHEET IS ONLY FOR STEEL PILES IN A COHESIONLESS SOIL.

24	If applicable, enter the strong axis radius of gyration (the strong axis is assumed parallel to the backwall.)
25	If applicable, enter the weak axis radius of gyration (the weak axis is assumed perpendicular to the backwall.)
26	Enter the vertical distance between the stream elevation and the superstructure bearing points.
27	Use the pull-down menu provided to select the type of lateral restraint system (if any) for this analysis.
28	If applicable, use the pull-down menu provided to select the anchor rod yield stress.
29	If applicable, enter the total number of anchor rods for one abutment.
30	If applicable, enter the anchor rod diameter.
31	If applicable, enter the height of the anchor block .
32	If applicable, enter the vertical distance between the stream elevation and the bottom of the anchor block.
33	If applicable, enter the anchor rod length for this analysis. This value must be greater than or equal to the value given directly above this input cell.

County:
Project No:
Description:

computed by:
checked by:
date: 7/21/2004

THIS WORKSHEET IS ONLY FOR STEEL PILES IN A COHESIONLESS SOIL.

Instructions Worksheet

Go to Pile and Soil Selection Worksheet

General Bridge Input	1	Span length	60.00	ft		
	2	Roadway width	24.00	ft		
	3	Location of exterior pile relative to the edge of the roadway	0.50	ft		
		Maximum number of piles	10	piles on	2.56	ft centers
		Minimum number of piles	4	piles on	7.67	ft centers
	4	Number of piles	6			
	5	Backwall height	6.00	ft		
	6	Estimated scour depth	2.00	ft		
	7	Superstructure system	PCDT			
		Estimated dead load abutment reaction	180.9	kip per abutment (default value)		
	8	Dead load abutment reaction for this analysis	180.9	kip per abutment		
		Estimated live load abutment reaction	121.5	kip per abutment (default value)		
	9	Live load abutment reaction for this analysis	121.5	kip per abutment		
Foundation Material Input	10	Soil SPT blow count (N)	20			
		Correlated soil friction angle (ϕ)	33.3	degrees		
	11	Soil friction angle for this analysis	33.3	degrees		
	12	Type of vertical pile bearing resistance	friction & end bearing			
	13	Estimated friction bearing value for depths less than 30 ft	0.7	tons per ft		
	14	Estimated friction bearing value for depths greater than 30 ft	0.8	tons per ft		
	15	Depth to adequate end bearing foundation material	40	ft		
	16	SPT blow count for end bearing foundation material	100 < N < 200			
Pile Input	17	Pile yield stress	36	ksi		
	18	Select pile type	HP10x42			
	19	Pile cross sectional area	12.4	sq. in.		
	20	Pile flange width	10.1	in.		
	21	Pile moment of inertia (strong axis)	210	in.^4		
	22	Pile section modulus (strong axis)	43.4	cubic in.		
	23	Pile section modulus (weak axis)	14.2	cubic in.		
	24	Pile radius of gyration (strong axis)	4.13	in.		
	25	Pile radius of gyration (weak axis)	2.41	in.		
Lateral Restraint Input	26	Superstructure bearing elevation	3.50	ft		
	27	Type lateral restraint system	buried concrete anchor block			
	28	Anchor rod yield stress	60	ksi		
	29	Total number of anchor rods per abutment	5	per abutment		
	30	Anchor rod diameter	0.75	in.		
	31	Height of anchor block	2.50	ft		
	32	Bottom elevation of anchor block	2.00	ft		
		Anchor block geotechnical capacity	8.11	kip per pile		
		Computed anchor force per pile	7.25	kip per pile		
		Minimum anchor rod length	12.15	ft		
	33	Anchor rod length for this analysis	14.00	ft		

County:
Project No:
Description:

computed by:
checked by:
date: 7/21/2004

THIS WORKSHEET IS ONLY FOR STEEL PILES IN A COHESIONLESS SOIL.

Check Pile Design

Geotechnical, Structural and Serviceability Requirements

Design Check	1	Axial pile stress	$P/A \le \sigma_{ALL}$	5.84 ksi	OK
	2	Pile bearing capacity	Axial Pile Load ≤ Capacity	111.6 kip	OK
	3	Interaction equation validation	$\frac{1}{(1-f_a/F'_e)} > 1.0$	1.03	OK
	4	Combined loading interaction requirement # 1	$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{(1-f_a/F'_{ex})F_{bx}} \le 1.0$	0.43	OK
	5	Combined loading interaction requirement # 2	$\frac{f_a}{0.60F_y} + \frac{f_{bx}}{F_{bx}} \le 1.0$	0.58	OK
	6	Anchor rod stress	$\sigma \le 0.6F_y$	19.7 ksi	OK
	7	Anchor block capacity	Total Anchor Force ≤ Capacity	8.1 kip per pile	OK
	8	Maximum displacement	$\delta_{MAX} \le 1.5$ in.	0.18 in.	OK

Anchor Design Worksheet (if applicable)

Foundation Summary	1	Roadway width	24.00 ft
	2	Span length	60.00 ft
	3	Distance between superstructure bearings and roadway grade	2.50 ft
	4	Backwall height	6.00 ft
	5	Dead load abutment reaction	180.9 kip per abutment
	6	Live load abutment reaction	121.5 kip per abutment
	7	Number of piles	6
	8	Total axial pile load	36.2 tons
	9	Pile spacing	4.60 ft
	10	Pile size	HP10x42
	11	Pile yield stress	36 ksi
	12	Minimum total pile length	44 ft

TIMBER PILES IN A COHESIVE SOIL
INSTRUCTIONS AND PILE DESIGN WORKSHEET

County:
Project No:
Description:

computed by:
checked by:
date: 7/21/2004

THIS WORKSHEET IS ONLY FOR TIMBER PILES IN A COHESIVE SOIL.

The calculations performed in the Pile Design Worksheet are based on the guidelines of the AASHTO Standard Specifications, the AISC Manual of Steel Construction, the Iowa DOT Bridge Design Manual (Iowa DOT BDM), and the National Design Specifications Manual for Wood Construction (NDS Manual).

Once the instructions on this worksheet have been reviewed, proceed to the Pile Design Worksheet or return to the pile and soil selection worksheet by clicking the icons below.

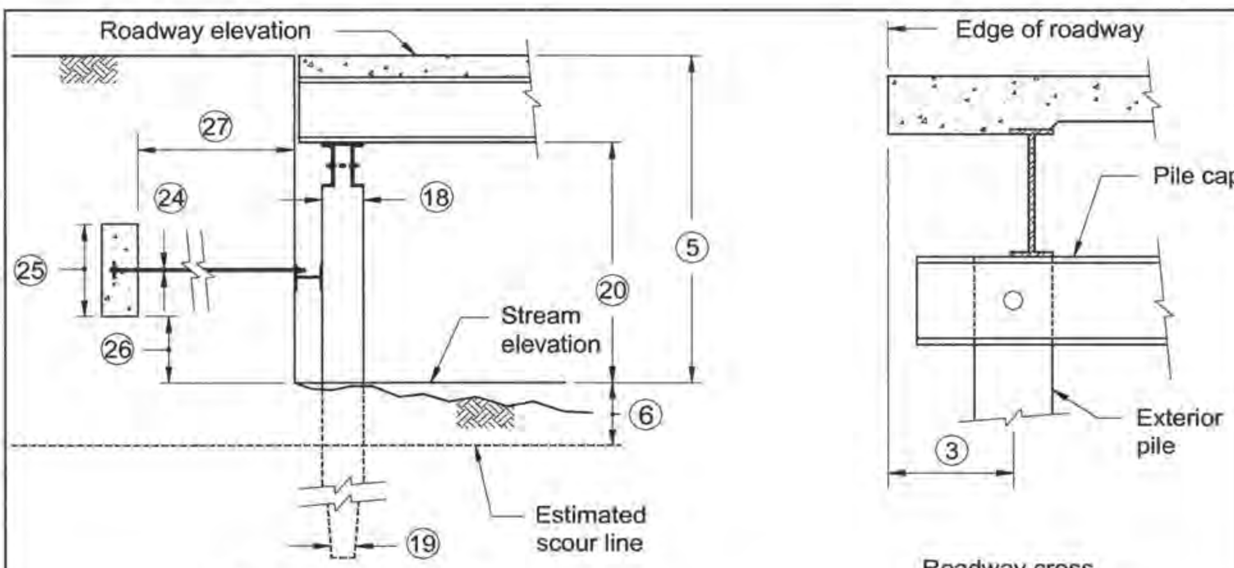
Pile Design Worksheet

Return to Pile and Soil Selection Worksheet

Data required is to be entered in the highlighted cells of the Pile Design Worksheet.

The stream elevation is the datum for all elevations.

The figure below is to be used as a reference for the vertical input dimensions.



Abutment cross section.

Roadway cross section near abutment.

County:
Project No:
Description:

computed by:
checked by:
date: 7/21/2004

THIS WORKSHEET IS ONLY FOR TIMBER PILES IN A COHESIVE SOIL.

The following numbers and explanations correspond the highlighted cells on the Pile Design Worksheet; all circled numbers are shown on the figure above.

Cell No.	Description
1	Enter the span length between the centerline of the abutment bearings.
2	Enter the roadway width of the bridge.
3	Enter the distance between the centerline of the exterior pile and the edge of the roadway. This value is positive for situations when the exterior pile is within the limits of the roadway width as shown above.
4	Enter the number of piles per abutment. This value must be within the range given in the cells directly above this input cell.
5	Enter the vertical distance between the roadway grade and the stream elevation.
6	Enter the vertical distance from the stream elevation to the estimated depth of scour. This value is based on stream hydraulics, geological information, and engineering judgment.
7	Use the pull-down menu provided to select the superstructure system for this analysis
8	Enter the dead load abutment reaction for this analysis. A default value maybe provided in the cell directly above this input cell.
9	Enter the live load abutment reaction for this analysis. A default value is provided in the cell directly above this input cell.
10	Enter the average standard penetration test (SPT) blow count (N-value) for the upper level soil.
11	Enter the undrained shear strength of the soil for this analysis. A default value base of the SPT N-value is provided in the cell located directly above this input cell.
12	Enter the friction bearing resistance per foot of pile for soils *within* 30 ft of the natural ground line. Appendix C of Volume I provides friction bearing values based on the SPT N-value.
13	Enter the friction bearing resistance per foot of pile for soils *not within* 30 ft of the natural ground line. Appendix C of Volume I provides friction bearing values based on the SPT N-value.
14	Use the pull-down menu provided to select the timber species for this analysis.
15	Enter the tabulated timber bending stress.
16	Enter the tabulated timber compressive stress (parallel to the grain).
17	Enter the tabulated modulus of elasticity.
18	Enter the pile butt diameter (i.e., the driving end).
19	Enter the pile tip diameter (i.e., the embedded end).
20	Enter the vertical distance between the stream elevation and the superstructure bearing points.
21	Use the pull-down menu provided to select the type of lateral restraint system (if any) for this analysis.
22	If applicable, use the pull-down menu provided to select the anchor rod yield stress.
23	If applicable, enter the total number of anchor rods for one abutment.
24	If applicable, enter the anchor rod diameter.
25	If applicable, enter the height of the anchor block.
26	If applicable, enter the vertical distance between the stream elevation and the bottom of the anchor block.
27	If applicable, enter the anchor rod length. This value must be greater than or equal the value given directly above this input cell.

County:
Project No:
Description:

computed by:
checked by:
date: 7/21/2004

THIS WORKSHEET IS ONLY FOR TIMBER PILES IN A COHESIVE SOIL.

Instructions Worksheet

Return to Pile and Soil Selection Worksheet

General Bridge Input	1	Span length	60.00 ft		
	2	Roadway width	24.00 ft		
	3	Location of exterior pile relative to the edge of the roadway	0.50 ft		
		Maximum number of piles	10	piles on	2.56 ft centers
		Minimum number of piles	4	piles on	7.67 ft centers
	4	Number of piles	9		
	5	Backwall height	6.00 ft		
	6	Estimated scour depth	2.00 ft		
	7	Superstructure system	PCDT		
		Estimated dead load abutment reaction	180.9 kip per abutment (default value)		
	8	Dead load abutment reaction for this analysis	180.9 kip per abutment		
		Estimated live load abutment reaction	121.5 kip per abutment (default value)		
	9	Live load abutment reaction for this analysis	121.5 kip per abutment		
Foundation Material Input	10	Soil SPT blow count (N)	10		
		Correlated soil un-drained shear strength (C_U)	1,270 psf		
	11	Soil undrained shear strength for this analysis	1,270 psf		
	12	Estimated friction bearing value for depths less than 30 ft	0.7 tons per ft		
	13	Estimated friction bearing value for depths greater than 30 ft	0.8 tons per ft		
Pile Input	14	Timber species	southern pine		
	15	Tabulated timber bending stress	1,750 psi		
	16	Tabulated timber compressive stress	1,100 psi		
	17	Tabulated timber modulus of elasticity	1,600,000 psi		
	18	Pile butt diameter	14.0 in.		
	19	Pile tip diameter	10.0 in.		
Lateral Restraint Input	20	Superstructure bearing elevation	3.50 ft		
	21	Type of lateral restraint system	buried concrete anchor block		
	22	Anchor rod yield stress	60 ksi		
	23	Total number of anchor rods per abutment	5 per abutment		
	24	Anchor rod diameter	0.75 in.		
	25	Height of anchor block	3.00 ft		
	26	Bottom elevation of anchor block	2.00 ft		
		Anchor block geotechnical capacity	5.5 kip per pile		
		Computed anchor force per pile	5.0 kip per pile		
		Minimum anchor rod length	13.08 ft		
	27	Anchor rod length for this analysis	14.00 ft		

Check Pile Design

County:
Project No:
Description:

computed by:
checked by:
date: 7/21/2004

THIS WORKSHEET IS ONLY FOR TIMBER PILES IN A COHESIVE SOIL.

Design Checks					
	1	Axial pile load	$\boxed{P \le P_{ALLOWABLE}}$	47.3 kip	OK
	2	Pile length	$\boxed{\text{Length} \le 55\text{ ft}}$	36 ft	OK
	3	Pile bearing capacity	$\boxed{\text{Axial Pile Load} \le \text{Capacity}}$	sufficient if pile is embedded at least	33 ft
	4	Interaction equation validation	$\boxed{\frac{1}{(1-f_C/F'_e)} > 1.0}$	1.04	OK
	5	Combined loading interaction requirement $\boxed{\left(\frac{f_C}{F'_C}\right)^2 + \frac{f_{bx}}{F'_b\left(1-\frac{f_C}{F'_{ex}}\right)} + \frac{f_{by}}{F'_b\left(1-\frac{f_C}{F'_{ey}}-\left(\frac{f_{bx}}{F'_{bE}}\right)^2\right)} \le 1.0}$		0.59	OK
	6	Anchor rod stress	$\boxed{\sigma \le 0.6F_y}$	20.2 ksi	OK
	7	Anchor block capacity	$\boxed{\text{Total Anchor Force} \le \text{Capacity}}$	5.5 kip per pile	OK
	8	Maximum displacement	$\boxed{\delta_{MAX} \le 1.5\text{ in.}}$	0.20 in.	OK

Anchor Design Worksheet (if applicable)

Foundation Summary				
	1	Roadway width		24.00 ft
	2	Span length		60.00 ft
	3	Distance between superstructure bearings and roadway grade		2.50 ft
	4	Backwall height		6.00 ft
	5	Dead load abutment reaction		180.9 kip per abutment
	6	Live load abutment reaction		121.5 kip per abutment
	7	Number of piles		9
	8	Total axial pile load		23.6 tons
	9	Pile spacing		2.88 ft
	10	Pile size		
			Butt diameter	14.0 in.
			Tip diameter	10.0 in.
	11	Pile material properties		
			Timber species	southern pine
			Tabulated timber compressive stress	1,100 psi
			Tabulated timber bending stress	1,750 psi
			Tabulated timber modulus of elasticity	1,600,000 psi
	12	Minimum total pile length		36 ft

TIMBER PILES IN A COHESIONLESS SOIL
INSTRUCTIONS AND PILE DESIGN WORKSHEET

County:
Project No:
Description:

computed by:
checked by:
date: 7/21/2004

THIS WORKSHEET IS ONLY FOR TIMBER PILES IN A COHESIONLESS SOIL.

The calculations performed in the Pile Design Worksheet are based on the guidelines of the AASHTO Standard Specifications, the AISC Manual of Steel Construction, the Iowa DOT Bridge Design Manual (Iowa DOT BDM), and the National Design Specifications Manual for Wood Construction (NDS Manual).

Once the instructions on this worksheet have been reviewed, proceed to the Pile Design Worksheet or return to the pile and soil selection worksheet by clicking the icons below.

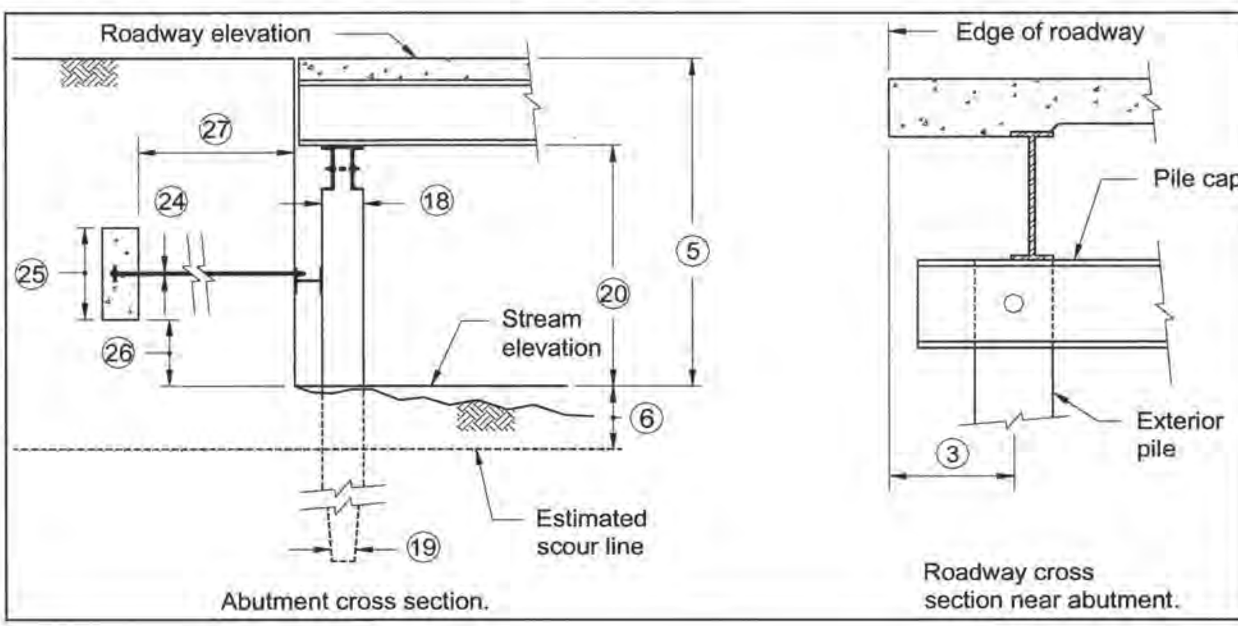
Pile Design Worksheet

Return to Pile and Soil Selection Worksheet

Data required is to be entered in the highlighted cells of the Pile Design Worksheet.

The stream elevation is the datum for all elevations.

The figure below is to be used as a reference for the vertical input dimensions.



County:
Project No:
Description:

computed by:
checked by:
date: 7/21/2004

THIS WORKSHEET IS ONLY FOR TIMBER PILES IN A COHESIONLESS SOIL.

The following numbers and explanations correspond the highlighted cells on the Pile Design Worksheet; all circled numbers are shown on the figure above.

Cell No.	Description
1	Enter the span length between the centerline of the abutment bearings.
2	Enter the roadway width of the bridge.
3	Enter the distance between the centerline of the exterior pile and the edge of the roadway. This value is positive for situations when the exterior pile is within the limits of the roadway width as shown above.
4	Enter the number of piles per abutment. This value must be within the range given in the cells directly above this input cell.
5	Enter the vertical distance between the roadway grade and the stream elevation.
6	Enter the vertical distance from the stream elevation to the estimated depth of scour. This value is based on stream hydraulics, geological information, and engineering judgment.
7	Use the pull-down menu provided to select the superstructure system for this analysis
8	Enter the dead load abutment reaction for this analysis. A default value maybe provided in the cell directly above this input cell.
9	Enter the live load abutment reaction for this analysis. A default value is provided in the cell directly above this input cell.
10	Enter the average standard penetration test (SPT) blow count (N-value) for the upper level soil.
11	The soil friction angle for this analysis. A default value is provided in the cell located directly above this input cell.
12	Enter the friction bearing resistance per foot of pile for soils *within* 30 ft of the natural ground line. Appendix C of Volume I provides friction bearing values based on the SPT N-value.
13	Enter the friction bearing resistance per foot of pile for soils *not within* 30 ft of the natural ground line. Appendix C of Volume I provides friction bearing values based on the SPT N-value.
14	Use the pull-down menu provided to select the timber species for this analysis.
15	Enter the tabulated timber bending stress.
16	Enter the tabulated timber compressive stress (parallel to the grain).
17	Enter the tabulated modulus of elasticity.
18	Enter the pile butt diameter (i.e., the driving end).
19	Enter the pile tip diameter (i.e., the embedded end).
20	Enter the vertical distance between the stream elevation and the superstructure bearing points.
21	Use the pull-down menu provided to select the type of lateral restraint system (if any) for this analysis.
22	If applicable, use the pull-down menu provided to select the anchor rod yield stress.
23	If applicable, enter the total number of anchor rods for one abutment.
24	If applicable, enter the anchor rod diameter.
25	If applicable, enter the height of the anchor block.
26	If applicable, enter the vertical distance between the stream elevation and the bottom of the anchor block.
27	If applicable, enter the anchor rod length. This value must be greater than or equal the value given directly above this input cell.

County:
Project No:
Description:

computed by:
checked by:
date: 7/21/2004

THIS WORKSHEET IS ONLY FOR TIMBER PILES IN A COHESIONLESS SOIL.

Instructions Worksheet

Return to Pile and Soil Selection Worksheet

General Bridge Input	1	Span length	60.00	ft	
	2	Roadway width	24.00	ft	
	3	Location of exterior pile relative to the edge of the roadway	0.92	ft	
		Maximum number of piles	9	piles on	2.77 ft centers
		Minimum number of piles	4	piles on	7.39 ft centers
	4	Number of piles	9		
	5	Backwall height	6.00	ft	
	6	Estimated scour depth	2.00	ft	
	7	Superstructure system	PCDT		
		Estimated dead load abutment reaction	180.9	kip per abutment (default value)	
	8	Dead load abutment reaction for this analysis	180.9	kip per abutment	
		Estimated live load abutment reaction	121.5	kip per abutment (default value)	
	9	Live load abutment reaction for this analysis	121.5	kip per abutment	
Foundation Material Input	10	Soil SPT blow count (N)	20		
		Correlated soil friction angle (ϕ)	33.3	degrees	
	11	Soil friction angle for this analysis	33.3	degrees	
	12	Estimated friction bearing value for depths less than 30 ft	0.7	tons per ft	
	13	Estimated friction bearing value for depths greater than 30 ft	0.7	tons per ft	
Pile Input	14	Timber species	southern pine		
	15	Tabulated timber bending stress	1,750	psi	
	16	Tabulated timber compressive stress	1,100	psi	
	17	Tabulated timber modulus of elasticity	1,600,000	psi	
	18	Pile butt diameter	13.0	in.	
	19	Pile tip diameter	10.0	in.	
Lateral Restraint Input	20	Superstructure bearing elevation	3.50	ft	
	21	Type of lateral restraint system	buried concrete anchor block		
	22	Anchor rod yield stress	60	ksi	
	23	Total number of anchor rods per abutment	5	per abutment	
	24	Anchor rod diameter	0.75	in.	
	25	Height of anchor block	3.00	ft	
	26	Bottom elevation of anchor block	2.00	ft	
		Anchor block geotechnical capacity	5.3	kip per pile	
		Computed anchor force per pile	4.9	kip per pile	
		Minimum anchor rod length	13.08	ft	
	27	Anchor rod length for this analysis	14.00	ft	

Check Pile Design

County:
Project No:
Description:

computed by:
checked by:
date: 7/21/2004

THIS WORKSHEET IS ONLY FOR TIMBER PILES IN A COHESIONLESS SOIL.

Design Checks					
	1	Axial pile load	$\boxed{P \le P_{ALLOWABLE}}$	47.3 kip	OK
	2	Pile length	$\boxed{\text{Length} \le 55 \text{ ft}}$	37 ft	OK
	3	Pile bearing capacity	$\boxed{\text{Axial Pile Load} \le \text{Capacity}}$	sufficient if pile is embedded at least	34 ft
	4	Interaction equation validation	$\boxed{\frac{1}{(1-f_C/F'_e)} > 1.0}$	1.04	OK
	5	Combined loading interaction requirment $\boxed{\left(\frac{f_C}{F'_C}\right)^2 + \frac{f_{bx}}{F'_b\left(1-\frac{f_C}{F'_{ex}}\right)} + \frac{f_{by}}{F'_b\left(1-\frac{f_C}{F'_{ey}}-\left(\frac{f_{bx}}{F'_{bE}}\right)^2\right)} \le 1.0}$		0.59	OK
	6	Anchor rod stress	$\boxed{\sigma \le 0.6F_y}$	20.0 ksi	OK
	7	Anchor block capacity	$\boxed{\text{Total Anchor Force} \le \text{Capacity}}$	5.3 kip per pile	OK
	8	Maximum displacement	$\boxed{\delta_{MAX} \le 1.5 \text{ in.}}$	0.20 in.	OK

Anchor Design
Worksheet
(if applicable)

Foundation Summary				
	1	Roadway width		24.00 ft
	2	Span length		60.00 ft
	3	Distance between superstructure bearings and roadway grade		2.50 ft
	4	Backwall height		6.00 ft
	5	Dead load abutment reaction		180.9 kip per abutment
	6	Live load abutment reaction		121.5 kip per abutment
	7	Number of piles		9
	8	Total axial pile load		23.6 tons
	9	Pile spacing		2.77 ft
	10	Pile size		
			Butt diameter	13.0 in.
			Tip diameter	10.0 in.
	11	Pile material properties		
			Timber species	southern pine
			Tabulated timber compressive stress	1,100 psi
			Tabulated timber bending stress	1,750 psi
			Tabulated timber modulus of elasticity	1,600,000 psi
	12	Minimum total pile length		37 ft

ANCHOR DESIGN WORKSHEET

County:
Project No:
Description:

computed by:
checked by:
date: 7/21/2004

THIS WORKSHEET IS ONLY TO BE USED AFTER THE PILE SYSTEM HAS BEEN DESIGNED AND ALL DESIGN REQUIREMENTS HAVE BEEN SATISFIED.

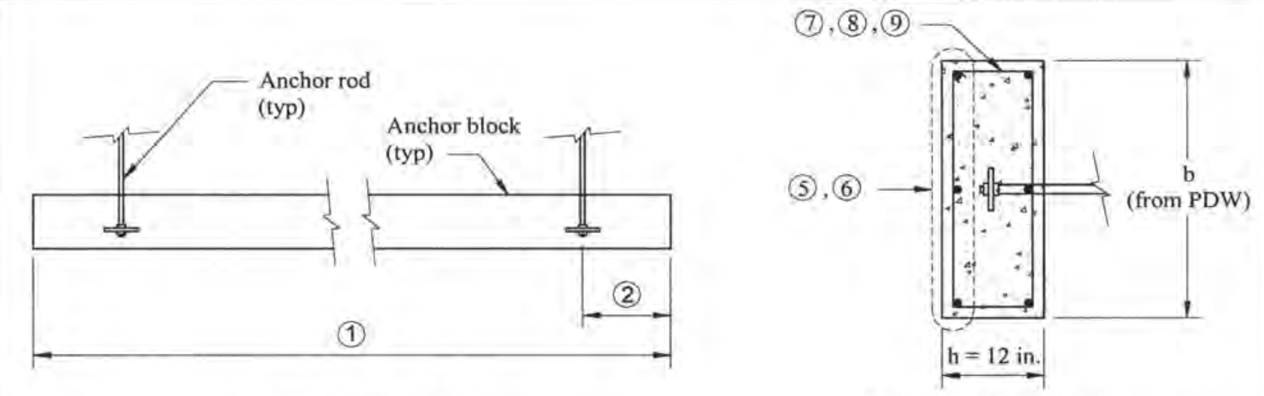
Return to Pile Design Worksheet

Go to Pile and Soil Selection Worksheet

The design in this worksheet is based on Section 8 of the AASHTO Standard Specifications.

Once the instructions on this sheet have been reviewed, proceed to the input section of this worksheet below.

Data required is to be entered in the highlighted cells of the Input Information section; all circled numbers are shown on the figure below.



Anchor block plan view.

Anchor block cross section.

Instructions	Cell No.	Description
	①	Enter the total length of the anchor block.
	②	Enter the distance between the end of the anchor block and the exterior anchor rod.
	3	Enter the anchor block concrete compressive strength.
	4	Use the pull-down menu provided to select the yield strength of the reinforcing steel.
	⑤	Enter the number of tension steel reinforcing bars on one vertical anchor block face.
	⑥	Use the pull-down menu provided to select the tension steel bar size.
	⑦	If applicable, use the pull-down menu provided to select the stirrup bar size.
	⑧	If applicable, enter the number of stirrup legs per section.
	⑨	If applicable, enter the stirrup spacing for this analysis. This value must be less than the value in the cell directly above this input cell.

County:
Project No:
Description:

computed by:
checked by:
date: 7/21/2004

THIS WORKSHEET IS ONLY TO BE USED AFTER THE PILE SYSTEM HAS BEEN DESIGNED AND ALL DESIGN REQUIREMENTS HAVE BEEN SATISFIED.

Input Information	1	Anchor block length		28.00 ft
	2	Distance from end of anchor block to exterior anchor rod		1.50 ft
	3	Concrete compressive strength		3.0 ksi
	4	Yield strength of reinforcing steel		60 ksi
		Determine tension steel area required for strength	$(A_S)_{REQ'D}$	
		Tension steel area required		0.22 in.^{2}
	5	Number of tension steel bars per vertical anchor block face		3 bars
	6	Tension steel bar size		4 #
		Tension steel area provided		0.59 in.^{2}
		Are stirrups required?		Yes
	7	Shear stirrup bar size number		3 #
	8	Number of stirrup legs per section		2
		Maximum stirrup spacing		4.69 in.
	9	Stirrup spacing for this analysis		4.50 in.

Design Checks	1	Design flexural capacity	$\boxed{M_U < \phi M_N}$	24.24 ft-kips	OK	{AASHTO 8.16.3.2}
	2	Reinforcement ratio	$\boxed{\rho < 0.75\rho_b}$	0.0021	OK	{AASHTO 8.16.3.2.2}
	3	Minimum reinforcement			OK	{AASHTO 8.17}
	4	Design shear capacity	$\boxed{V_U < \phi V_N}$	49.7 kip	OK	{AASHTO 8.16.6.1.1}

Anchor System Summary	1	Number of anchor rods		5		
	2	Anchor rod yield stress		60 ksi		
	3	Anchor rod diameter		0.750 in.		
	4	Anchor rod length		14.00 ft		
	5	Anchor rod spacing		6.25 ft		
	6	Vertical distance between bottom of anchor block and roadway grade		4.00 ft		
	7	Anchor block length		28.00 ft		
	8	Anchor block height		2.5 ft		
	9	Anchor block width		12.0 in.		
	10	Concrete compressive strength		3.0 ksi		
	11	Details for reinforcement on vertical anchor block face		3	#	4 bars
	12	Details for shear stirrups	#	3 bars on		4.50 in. centers

APPENDIX D

GENERIC STANDARD ABUTMENT PLANS

D.1. GENERAL INFORMATION

These generic standard abutment design sheets were developed to provide the user with a means of producing a set of drawings for a single span stub abutment in the 20 to 90 ft range with no or small skew angles. By using the FDT and inserting basic geometry and job information, the designer can generate a complete set of abutment construction drawings.

Although an effort has been made to give sufficiently complete information and to allow for adaptation to specific sites, requirements imposed by site conditions may necessitate modification of these drawings.

The completed set of abutment drawings assembled from these templates shall be reviewed and approved by a Registered Professional Engineer prior to the beginning of construction. It is important that a subsurface soil investigation be performed prior to completion of the foundation design and drawings. It is recommended, whenever possible, that a SPT be performed. Howeever, a more accurate foundation design can be completed if the soil undrained shear strength or friction angle is determined for cohesive and cohesionless soils, respectively. These parameters can then be used in the FDT.

The concepts, designs, details, and notes shown in these standard plans for the piles and anchor system have been developed by the BEC of Iowa State University using the guidelines specified in the AASHTO Standard Specifications, the AISC Manual, the NDS manual, the Iowa DOT BDM and proven design practices. While the bridge system shown has been carefully designed, detailed, and checked, any user should independently determine appropriateness, and potential adaptability of this design methodology for the abutments of a specific bridge site.

D.2. INSTRUCTIONS FOR USING CONSTRUCTION DRAWINGS

Prior to utilizing these drawings, the designer must obtain basic survey and geometric data for the proposed construction site. Information concerning the foundation material and the elevation of the potential bearing areas must also be obtained.

Once the design has been completed and all necessary geometry, bearing elevations, finished ground elevations, etc. have been determined, the designer can produce the final construction drawings, an example of which is included in this appendix. A complete set of construction drawings is provided in the TR-486 final report. Completed drawings should be included with the final set of construction documents. The following steps should be followed in the preparation process:

1. Complete the superstructure design.
2. Fill in all information pertinent to the bridge and construction site in indicated locations (i.e., fill in all boxes) including:
   - Basic survey information
   - Design details provided by the FDT.
3. Add drawing titles and add miscellaneous information including:
   - Customizing the standard drawings by adding necessary location and route information to the title block of each sheet.
   - Add necessary information pertaining to utilities, hydraulic data.
   - Add subsurface exploration data

The standard abutment plans provided consist of two different types of sheets. The first type consists of two general sheets that will be used for all bridge abutments and are both included in the final set of construction sheets. These include the cover sheet (Sheet 1) and a general bridge plan and elevation layout sheet (Sheet 2). The second type of sheets consists of a series of construction sheets (Sheets 3a, 3b, etc.) with different combinations of pile caps, backwall systems, anchor systems, and pile types. For example, if the bridge site requires steel H-piles with an anchor system, a concrete pile cap, and a sheet pile backwall, Sheet 3g should be used. At most, two of these construction sheets will be required for a particular bridge site (i.e., a different construction sheet for each bridge abutment). If the two bridge abutments use the same combination of previously mentioned substructure variables, the same sheet can be used twice with different dimensions, if necessary.