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Integral Abutment Bridge Design Guidelines

by the IAC

Design Criteria Including:

Structural
Geometrical
Hydraulic
Geotechnical

VTrans Structures Section
Montpelier, Vermont
VTrans Integral Abutment Committee

Second Edition

Integral Abutment Bridge Design Guidelines

SECOND EDITION

by

VTrans, Integral Abutment Committee

VTrans, Structures Section
Montpelier, Vermont

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ERRATA

An explanation for each correction is provided as a footnote at the appropriate section.

Section	Date	Description
6.2	06/12/2008	Clarified the material specifications for the anchorage.
Design Example	10/01/2008	Corrected an error in the design example. Equation 7-28 is not required and equation 7-27 has been corrected.
4.4.1 5.2 6.2	07/15/2009	Changed Bridge End Selection flow chart to match 2009 VTrans LRFD Structures Manual and changed all detail titles that read Type "J" to Type A to match the 2009 VTrans LRFD Structures Manual

TABLE OF CONTENTS

INTRODUCTION

This guideline supersedes the 2004 Integral Abutment Bridge Design Report published by the VTrans Structures Section. The guideline is intended to be used in conjunction with the new 2009 edition of LRFD Structures Manual to be published in 2009. Each chapter corresponds with the chapters of the LRFD Structures Manual. Where the content of this guideline is in conflict with the LRFD Structures Manual, this guideline shall be considered controlling.

Other than the new format of the guideline the following additional changes have been made to the 2004 Integral Abutment Bridge Design Report. These changes include:

1. The basic principal of the 2004 Report was that integral abutment bridges would be allowed only under certain criteria. Since then, more information has been collected that would suggest that integral abutment bridges can be used in almost every case. A modified list of criterion as presented in the 2004 Report has been retained to provide a basis for a Simplified Design Method. Any bridge not meeting the criteria would require additional design considerations not spelled out in the guideline.
2. A new section that provides guidance on laying out an integral abutment bridge has been added.
3. A new Loads chapter has been added to aid in modeling an integral abutment bridge for analysis.
4. Bridge end details have been refined due to experience gained over the years. Prestressed, precast girder information has been added.
5. The entire guideline has been converted to reflect the AASHTO LRFD Bridge Design Specifications (LRFD). A simplified pile selection is now provided. The intent is that the requirements of the LRFD Specifications will be met by using the recommendations in this guideline.
6. The guideline suggests the pile orientation be weak-axis bending.
7. The entire guideline has been converted to the L-Pile analysis software. Future editions of the guideline will include additional information covering the FB-Pier analysis software.
8. The 2004 Report limited the maximum moment of the pile to the limits provided by the AASHTO Standard Specifications for Highway Bridge Design. The guideline recognizes that the pile moment at the pile cap may go into the plastic region. This will be an allowable condition in the design. The LRFD steel column design considerations will apply (see LRFD Section 6.9).
9. A new section has been added to the guideline to help navigate through the requirements of Section 10 of the LRFD Specifications.
10. A pile design example reflecting the procedures in the guideline is included in the appendix.

Throughout this guideline, the term LRFD, when followed by a section or table number, is used as a reference to the 2007 AASHTO LRFD Bridge Design Specifications 4th edition complete with the 2009 interims.

BACKGROUND

Integral abutment (or jointless) bridges have a demonstrated history of initial cost savings due to economical use of materials and life cycle cost savings through reduced maintenance. Integral abutment bridges are being used to eliminate expansion joints at abutments and in bridge decks. This reduces both initial construction costs and continual maintenance costs. Because of this, designers are using integral abutment bridges more often throughout the United States. Designers in Tennessee have designed highway bridges up to several hundred feet with no joints, with the longest concrete bridge being 927 ft (*282 m*) and the longest steel bridge being 416 ft (*127 m*). A review of the National Bridge Inventory System (NBIS) in 1992 showed that 80% of NBIS bridges were less than 180 ft (*55 m*). The review also showed that 90% of these bridges were less than 400 ft (*122 m*). In Vermont, these percentages are even higher. One can see the advantage jointless bridge construction has for Vermont and the United States.

The Structures Section of VTrans has begun designing integral abutment bridges with the goal of decreasing the costs of replacing Vermont's bridges, while increasing the durability and longevity of these structures. To gain a greater understanding of jointless construction, the Structures Section along with the Geotechnical Section began a dialog that resulted in this report. This report is a culmination of information gathered from every aspect of bridge analysis and design. It brings the thoughts from those involved in the Geotechnical, Construction, Hydraulics and Structural fields together to create a general reference of integral abutment bridge design. VTrans intends this report to be used as a guide, and as such, each user is cautioned to use it with full engineering judgment. In cooperation with Contract Administration, the Structures Subcommittee modified the pile specifications to include the unique needs integral abutment bridges require.

Updates from further research will supplement this report. VTrans recently completed a research contract with Wiss, Janney, Elstner Associates, Inc. to study integral abutment bridges. The research involved a literature search to attempt to answer questions regarding the design and performance of integral abutment bridges. Presently, the agency has entered into a new research contract with the University of Massachusetts, Amherst (UMASS). This research will build upon WJE's research and take this efforts one more step forward. The UMASS research will develop an instrumentation plan and monitor the performance of a curved girder integral abutment bridge. In addition to this curved girder structure, UMASS will revise the instrumentation plans developed by WJE for two straight girder integral abutment bridges. UMASS will also be responsible for the performance monitoring of these structures.

The information obtained from this valuable research will ensure that VTrans continues to design durable and cost effective structures while limiting environmental impacts.

SECTION 1 INTRODUCTION TO INTEGRAL ABUTMENT BRIDGES

1.1 INTEGRAL ABUTMENT BRIDGE

Integral abutment bridges are single span or multiple span continuous deck type structures with each abutment monolithically connected to the superstructure and supported by a single row of flexible piles. The primary purpose of monolithic construction is to eliminate the need for deck movement joints and bearings at abutments.

1.2 DIFFERENCE FROM CONVENTIONAL BRIDGES

The primary difference between an integral abutment bridge and a conventional bridge is the manner in which movement is accommodated. A conventional bridge accommodates movement by means of sliding bearing surfaces. An integral abutment bridge accommodates movement by designing each abutment to move unrestricted as a result of longitudinal loading effects with less induced stress, thus permitting the use of lighter and smaller abutments.

1.3 DOCUMENT PRECEDENCE

The content of this design guide supersedes the Structures Manual. The Structures Manual may be referred to for all other content not found in this guide.

1.4 DEFINITIONS

Abutment – A support at each end of a bridge.

Abutment Stem – Is comprised of a Pile Cap topped by a backwall.

Askew – The angle between the centerline of bearing and the centerline of the highway. (*See Skew*)

Backwall – Typically the second placement of concrete in an integral abutment. This segment of the abutment sets on top of the pile cap and is the segment the girders are embedded into.

Continuity Connection – A monolithic connection between two separate reinforced concrete components.

Flared Wingwalls – Wingwalls that extend from the abutment at an angle until the slope of the earth rising from the river or underpass meets the slope descending from the roadway. (See **Figure 2.2.1-1**)

Frame Action – Occurs when each end of a beam is fully embedded in its supports. Negative end moments from composite dead load and live load form along with positive mid-span moments.

In-Line Wingwalls – Short extensions off the abutment at either end. These extensions are in line with the abutment or pile cap. (See **Figure 2.2.1-1**)

Integral Abutment – An abutment comprised of a pile cap with an embedded superstructure, supported by a single line of piles.

Leveling Plate – A steel bearing plate that supports one end of a girder – also called a sole plate. This plate is supported by two large anchor bolts on either side of the girder. The plate's elevation can be field adjusted by raising, or lowering the nuts supporting it.

Lower Zone – The lower portion of the pile that is fully supported by earth along its length where any deflection is negligible.

Negative Moment Reinforcement – Requires steel reinforcement to resist the negative moment caused by deck loads at the abutments.

Nominal Axial Pile Resistance (NAPR) – The required strength of a pile, based on applied loads, adjusted by the resistance factor for axial strength in a pile.

Nominal Pile Driving Resistance (NPDR) – The required strength to drive a pile. Also called the geotechnical resistance of the pile.

Nominal Structural Pile Resistance (NSPR) – The axial structural strength of the pile defined by the pile section properties and the strength of the steel.

Pile Cap – A large prismatic volume of reinforced concrete topping a line of embedded piles. Typically the first placement of concrete in an integral abutment.

Pile Head – The top of the pile as it becomes embedded into the pile cap.

Pile Orientation – The direction a pile will be driven to counteract lateral deflections at the pile head. A pile can be oriented for weak axis bending or strong axis bending. (See **Figure 4.5.1.6-1**)

Pile Tip – The bottom most point of the pile. Typically sets on bedrock.

Plastic Hinge – The state of a steel section when an applied moment causes permanent deformation at a specific point. At this state, the cross section is either in full compressive or tensile failure. The boundary of these two failure zones is the neutral axis. Compact sections can maintain this state at a constant resistance throughout a certain deflection before the resistance starts to diminish.

Pre-bore – This is the process of excavating the top strata of rocky or otherwise rigid earth by various means. The purpose of pre-boring is to control the soil condition surrounding the upper zone of the pile, allowing it to deflect as required without rigid soils seizing it up.

Simplified Design Method – A design methodology presented in this guide that simplifies the design process for integral abutments by using general assumptions in the way an integral abutment bridge performs. (See section 2.2.1)

Simply Supported – A beam supported by a pin at one end and a roller at the other end. The beam is supported vertically and laterally with no other restrictions. Beam ends pivot at their supports therefore forming moments towards the center of the beam.

Skew – The angle between the centerline of bearing and an imaginary transverse line 90° to the centerline. (*See Askew*)

Strong Axis Bending - Bending a section about the axis that provides the most bending resistance. For I sections (such as H-piles), this typically means bending in the axis parallel to the flanges. (See **Figure 4.5.1.6-1**)

Thalweg – The path which water flows in a river at the highest velocity. This path generally follows the deepest profile of the river bed. This path does not necessarily run parallel with the sides of the channel.

Total Allowable Movement - The maximum allowed longitudinal movement at the abutment caused by expansion or contraction from shrinkage, creep and temperature effects in the deck.

U-Wall – An abutment with parallel wingwalls in line with the edge of the roadway. The abutment and wingwall configuration forms a "U" shape. (See **Figure 2.2.1-1**)

Unbraced Length – A length of a column that is not laterally braced by any sort of support.

Upper Zone – The top portion of the pile that has detectable deflections due to bending.

Weak Axis Bending – Bending a section about the axis that provides the least bending resistance. For I sections (such as H-piles), this typically means bending in the axis parallel to the web. (See **Figure 4.5.1.6-1**)

1.5 NOTATION

A_s = area of steel pile.
C = preliminary design factor used to select a pile.
DD = down drag.
F_y = steel strength of pile.
K = column effective length coefficient.
l_b = unbraced length.
M_p = plastic moment of a steel pile.
$M_{p'}$ = moment that creates a plastic hinge at the pile head with an axial load applied.
M_r = flexural resistance of the pile.
M_u = nominal flexural resistance of the pile.
M_u = applied factored Moment at the pile head.
P_n = Nominal Structural Pile Resistance (NSPR).
P_r = Structural Pile Resistance (SPR)
P_u = applied factored axial loads.
Q = design flow.
R_n = Nominal Axial Pile Resistance (NAPR).
R_{ndr} = Nominal Pile Driving Resistance (NPDR).
δ = lateral deflection of the pile head.
λ = normalized column slenderness factor.
σ_{dr} = maximum driving stress.
ϕ = resistance factor for driving a pile.
ϕ_c = resistance factor for compression in a pile.
ϕ_{da} = resistance factor for driving a pile.

ϕ_{dyn} = resistance factor for dynamic pile driving monitoring.
ϕ_f = resistance factor for flexure in a pile.
ϕ_{mon} = resistance factor based on pile driving monitoring.
ϕ_{stat} = resistance factor for static analysis pile driving monitoring.

SECTION 2 GENERAL DESIGN AND LOCATION FEATURES

2.1 FIRST CHOICE

Integral abutment construction shall be considered as a first option for all slab and slab on stringer bridges.

2.2 STRUCTURE GEOMETRICAL CRITERIA

States and Municipalities have successfully used integral abutment bridges as an effective means of eliminating bridge joints. Designs have ranged from simple (such as presented in this guideline), to very complex. The entire collection of the nation's integral abutment structures have encountered and overcome nearly all design challenges successfully. Though most integral abutment designs will not encounter such complexities, when such cases arise, engineering judgment and experience should be relied upon to find a solution. Using this guideline will aid the designer in designing the majority of the integral abutment bridges encountered.

The *Simplified Design Method* presented in Section 4.3.1 is based on the criteria listed in Section 2.2.1. These criteria shall not limit the use of the integral abutment method of bridge construction. Though much of the guideline applies to all integral abutment bridges; designs that exceed the limits in Section 2.2.1 may require a more detailed analysis and design (see Section 2.2.2).

2.2.1 Criteria for the Simplified Design Method

In order to use the *Simplified Design Method*, the structure:

- shall have a skew angle less than or equal to 20 degrees;
- shall be a straight bridge or a curved bridge with straight beams with all beams parallel with each other;
- shall use grade 50 (345) steel for H-Piles;
- shall use H-piles with a flange of 10 inches (254 mm) or larger;
- shall set upon parallel abutments and piers;
- shall have a maximum abutment height of 13 ft (4 m) to finished grade to reduce the passive earth pressure acting against each abutment (the designer should strive to use equal abutment heights at each end of the bridge);
- shall have a maximum total bridge length, as measured between centerlines of bearing at each abutment of:
  - 395 feet (119 m) for steel bridges, and
  - 695 feet (210 m) for concrete bridges;
- should have an individual span length between supports less than 145 feet (*44 m*);
- shall have a longitudinal slope of the bridge deck equal to or less than 5%;
- should have abutments with parallel wingwalls (otherwise called u-walls) (flared wingwalls may be considered at the discretion of the designer; when very short wingwalls are required, in-line wingwalls may also be considered) (See **Figure 2.2.1-1**.);
- shall have monolithic cantilevered wingwalls with a length of 10 ft (*3 m*) or less, as measured from the back face of the abutment. The portion of the wall beyond 10 ft (*3 m*) shall be designed as a freestanding retaining wall. An expansion joint should be detailed between the free standing retaining wall and the cantilevered wingwall to allow for up to 2 inch (*50 mm*) of movement; and
- shall have a minimum pile embedment length below the bottom of the pile cap of 16 ft (*5 m*) for design.

Any bridge that falls outside of the above requirements will require a more detailed design. See Section 4 for more information regarding both the *Simplified Design Method* and what is required for a more detailed design.

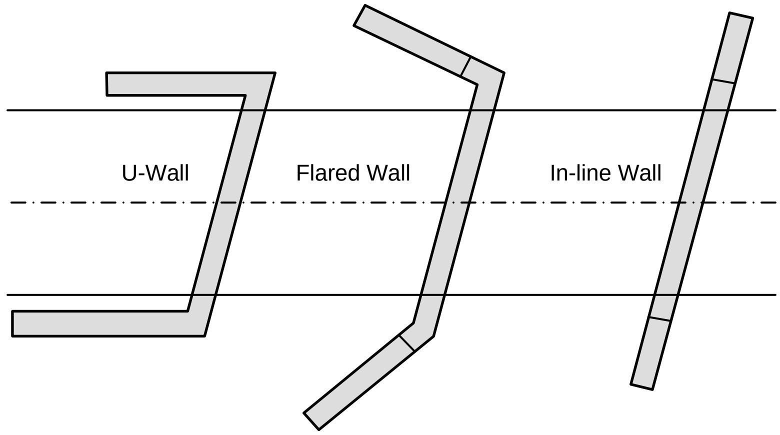

Figure 2.2.1-1 Parallel Wingwall or U-Wall Configuration.

2.2.2 Detailed Design for Projects That Exceed the Criteria for Simplified Design

A structure that falls outside of the criteria in Section 2.2.1 may require more detailed analysis. Typical situations where exceeding the criteria for the *Simplified Design Method* will require additional design considerations include:

- Longer spans will cause greater end rotations and the additional dead load may require enhancing the bearing details shown in the guideline.
- Curved structures will cause torsion. This torsion will be resisted at the girder ends. During construction, curved girders undergo varied bearing conditions which need to be addressed. For example, as girders are added to the deck system, working from the inside to the outside of the curve, an uplifting of the bearings on the inside girders can occur. The wet deck placement seems to enhance this effect. Curved girder decks are analyzed using a finite element method. Separate analysis should be done for each construction stage. A result of this analysis and design process suggests a different set of assumptions than provided in Section 2.2.1 will need to be utilized for design.
- Skews over 20° may cause the entire bridge to rotate in plan. This rotation may cause cracking in the pavement at the ends of the deck.

The designer should lay out the bridge according to the need of the locality and at the same time strive to keep within the requirements of the *Simplified Design Method*. If the geometry and other requirements for the bridge's locality require exceeding the criteria provided in Section 2.2.1, the designer should document their intention on how to proceed with the design. At this point the designer may choose to design an integral abutment bridge with a more detailed analysis or choose another structural alternative (see Section 2.2.3).

2.2.3 Semi-Integral, Jointed and Other Structural Alternatives

Situations may arise where a traditional integral abutment solution is not available. Before proceeding with a non-integral abutment alternative, the designer should document why an integral abutment solution was not utilized and why an alternative is justified. The designer may then consider the following alternatives:

Use semi-integral end details. This alternative has been used by VTrans for decades. These structures typically use a traditional footing-abutment substructure which may have high environmental impacts and construction costs. Refer to the Structures Manual for more information.

A hybrid solution may be explored, such as using an integral abutment at one end of the bridge and another system at the other. This may be necessary where a bridge is in close proximity of an intersection.

As a last resort, the designer may choose a fully jointed system. Jointed systems have negative attributes such as:

- high cost of constructing bridge joints,
- high cost of maintaining bridge joints,
- joints have proven to fail on most jointed bridges, and
- joints are typically used on bridges with typical footing-abutment substructures.

This choice should be limited to locations where all possible solutions for an integral abutment structure or other alternatives have been fully explored and found unfeasible.

2.3 LAYING OUT THE BRIDGE

During the scoping process, the designer will need to lay out several alternatives for the crossing. The purpose of this study is to minimize costs and resource impacts. When these ideals are in conflict, the designer will need to measure the appropriate balance between the two.

At times, the designer may be required to choose a longer structure to satisfy a requirement for limited impacts. At other sites, right of way impacts, construction scheduling, costs or material capabilities may dictate a shorter span, thereby increasing environmental impacts.

The following sections may be used to aid the designer with some basic principals of laying out an integral abutment structure. Using the following suggestions will result in ideal bridge geometry for the *Simplified Design Method*. Outside considerations may affect this ideal with strict elevation or length requirements. Due to project requirements, modifications to the proposed geometry will likely be required throughout the design phases.

2.3.1 Conventional Layout

Typically, geometrical considerations for integral abutment bridges cause a larger then needed hydraulic opening as deemed necessary by the Hydraulics Section. However, where the local conditions, bridge geometry or other considerations dictate that the bridge design maintains the minimum opening provided by Hydraulics, the bridge may be laid out in a conventional manner (See **Figure 2.3.1-1**). With this method, the resulting abutment height may be taller then recommended for the *Simplified Design Method*. If this is so, appropriate design considerations with regards to abutment height will need to be addressed.

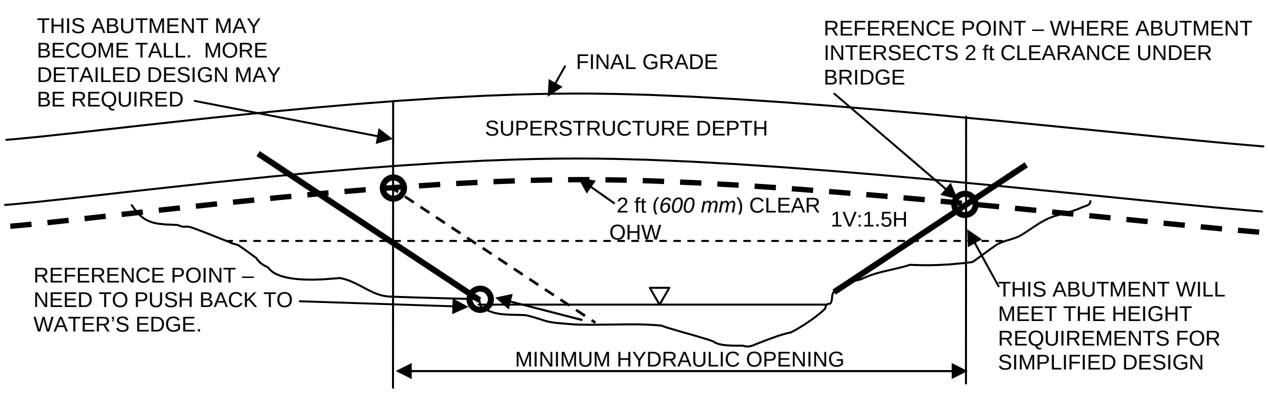

Figure 2.3.1-1 Reference points for Conventional Layout.

2.3.2 Ideal Layout

The designer should set a horizontal and vertical alignment of the roadway before laying out the structure.

From Section 2.2.1, the abutments and piers need to be parallel to each other and have a skew less than or equal to 20°. To keep the bridge length to a minimum, the abutments should be in line with the thalweg of the river. Extending the bridge length reduces the importance of this. Though this guideline emphasizes the priority in keeping the bridge as short as feasibly possible; keep in mind, holding the skew to 20° or less and limiting the height of the abutments will make designing, detailing, constructing and finally maintaining the structure simpler.

2.3.2.1 Channel Slopes Equal to or Flatter Than 1:1.5.

To begin laying out the structure, the designer will need to determine the existing slope of each river bank. Where the bridge is to span the entire floodplain or floodway, the designer will need to determine where the theoretical banks of the floodplain or floodway are. The designer will draw a line with an inclined slope of 1 vertical on 1.5 horizontal (1:1.5) from the reference point away from the channel at each abutment location (see Figure 2.3.1-1). This slope will be the proposed channel slope and the final surface of the stone fill.

If the river bank has a slope of 1:1.5 or flatter, then the reference point is where the ordinary high water (OHW) intersects with the slope (see **Figure 2.3.2.1-1**). This layout avoids construction impacts below the OHW elevation.

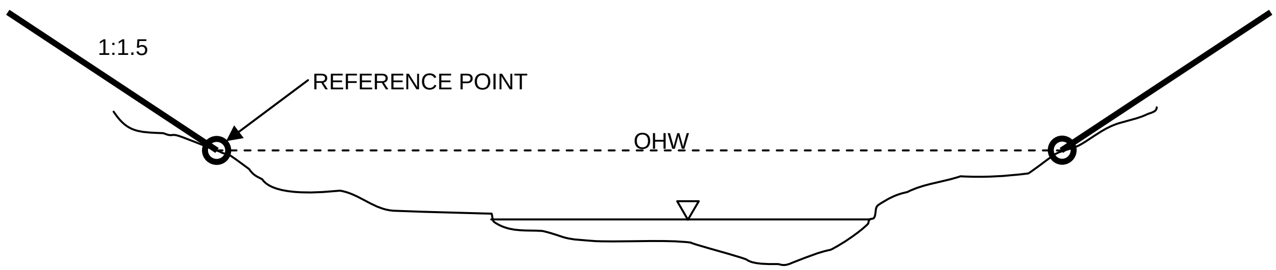

Figure 2.3.2.1-1 Reference points for slopes flatter than or equal to 1:1.5.

2.3.2.2 Channel Slopes Steeper Than 1:1.5.

Slopes steeper than 1:1.5, and bridges requiring a certain minimum span length need to be reinforced with a mechanical stabilized earth system with keyed-in stone fill in front of the abutment location to preserve the existing river bank slope (see **Figure 2.3.2.2-**). The reference point in these cases will be at the top of the river bank. This option should be considered only when span lengths shall be kept to a minimum.

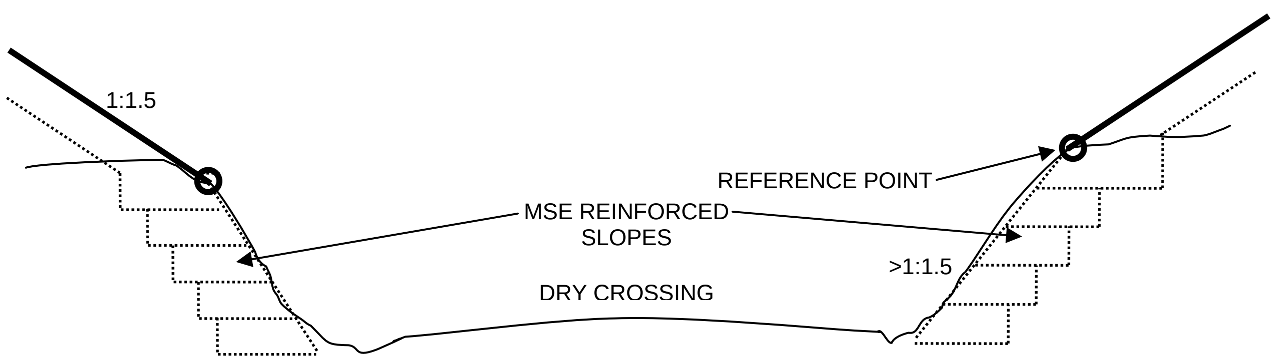

Figure 2.3.2.2-1 To minimize span for dry crossings, use reference points for slopes steeper than 1:1.5.

2.3.2.3 Making Structure Longer to Avoid Problems

Where river bank stability is of concern or river alignment with relation to roadway causes sharp skews and the bridge span length is not a concern, the reference point shall be at the toe of slope in the channel (see **Figure 2.3.2.3-1**).

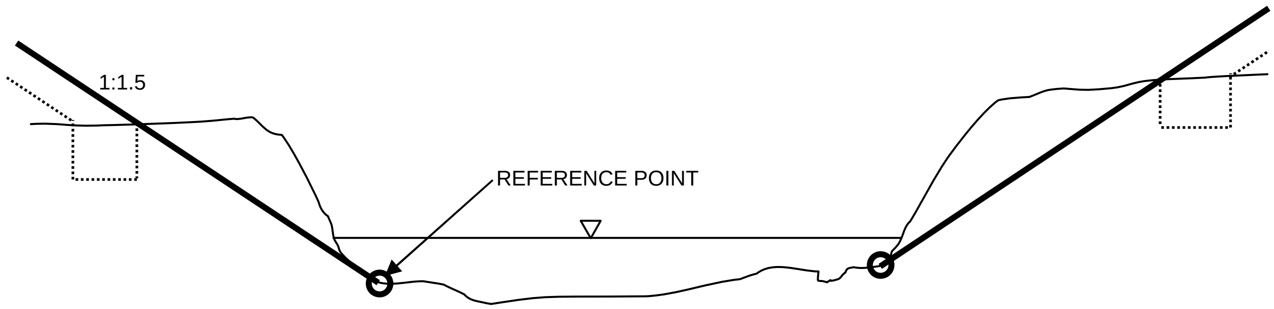

Figure 2.3.2.3-1 Reference points for slopes steeper than 1:1.5 where bank stability may be a concern.

2.3.2.4 Defining Bridge Geometry

To lay out the abutments, draw a line parallel of the roadway profile's final grade, which is offset by estimated depth of the superstructure, plus an additional 2 ft (*600 mm*) minimum clearance below the superstructure. This line is represented by the bold dashed line in **Figure 2.3.2.1-1**. This offset line will intersect the proposed channel slopes at the faces of the abutments. The distance between each abutment face plus the thickness of both abutments will be the length of the proposed structure (see **Figure 2.3.2.4-1**). This new length may be used to get a better estimated depth of structure which in turn can be used for another iteration of this process.

The intersection of the proposed channel slope and the final grade constitutes the theoretical ends of the parallel wingwalls (see **Figure 2.3.2.4-1**).

The minimum depth of the abutment shall extend to the bottom of stone fill (**see Figure 2.4-1**).

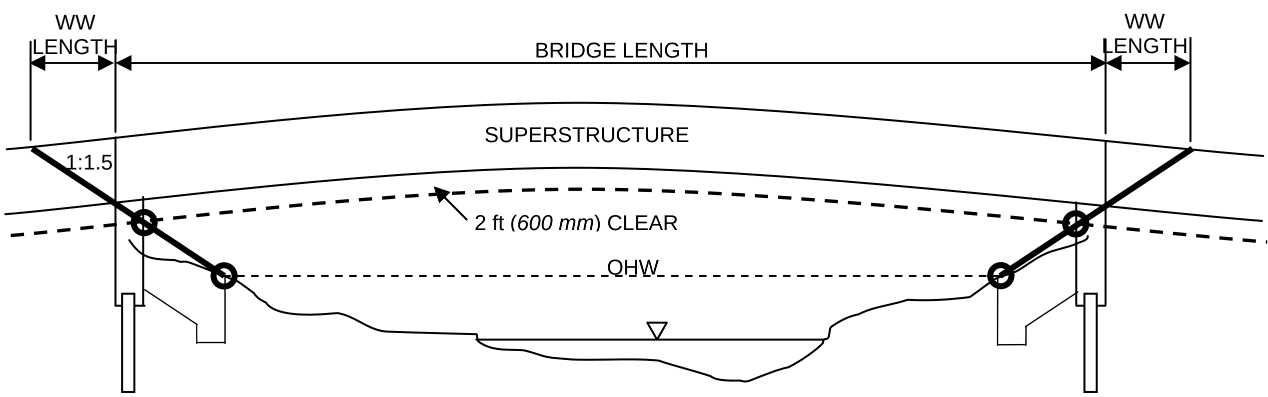

Figure 2.3.2.4-1 Defining Bridge Geometry from proposed channel slopes with parallel wingwalls.

2.4 HYDRAULIC REQUIREMENTS

The designer should refer to the Hydraulic Report and **Figure 2.4-1** for more information regarding the following:

- At a minimum, the bottom of the pile cap (*or abutment stem*) should extend below the bottom of stone fill.
- Piles should be designed as freestanding above the deeper elevation of 6 ft (*2 m*) below channel bottom or the projected contraction scour depth.
- The abutment setback distance is constrained by channel stability, potential for lateral migration, channel alignment, and susceptibility to scour. The severity of these constraints may govern structure choice at these locations. Use of an integral abutment bridge is not recommended at bends in the stream. Other site constraints that may restrict the abutment setback distance include: buildings, intersections, or existing retaining walls within close proximity to the existing channel.
- Maintain a minimum of 1 ft (*300 mm*) of freeboard at design flow (*Q*). Since integral abutment bridges are typically longer than those designed on spread footings, the depth of the beams may increase. Where

hydraulic clearance is a problem, the grade may need to be adjusted or venting holes may need to be specified on the up-most elevation of the girder webs.

2.4.1 Scour Considerations

The designer shall ensure the stability of the structure once the anticipated scour, as defined by the Hydraulics Section, has occurred. This may require driving the piles deeper then what is required by the criteria listed in this section.

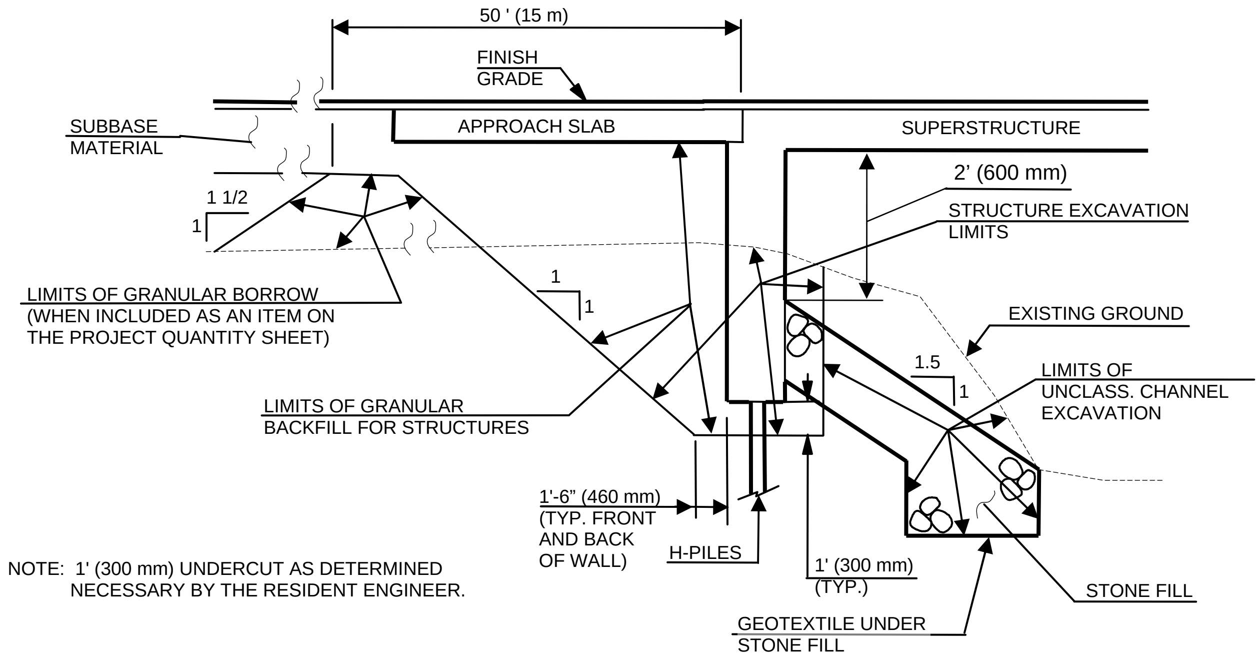

Figure 2.4-1 Earthworks Typical

2.4.2 Cofferdam Requirements

Based on requirements from the Stream Alteration, the Hydraulic and/or the Geotechnical Engineer, a cofferdam may be necessary to drive piles, or construct the pile cap. In cases where construction is required below ordinary high water (OHW), cofferdams should be considered to keep the construction in the dry.

2.5 GEOTECHNICAL

The designer should refer to the Geotechnical Report and Figure 2.5-1 for more information regarding the following:

- To ensure proper pile placement in difficult pile driving conditions, pre-excavating to a depth of 8 ft (*2.5 m*) below the bottom of pile cap elevation is recommended.
- Integral abutments located in soil conditions where anticipated settlements may occur, the designer shall consider additional effects not normally considered for the Simplified Design Method, such as downdrag.

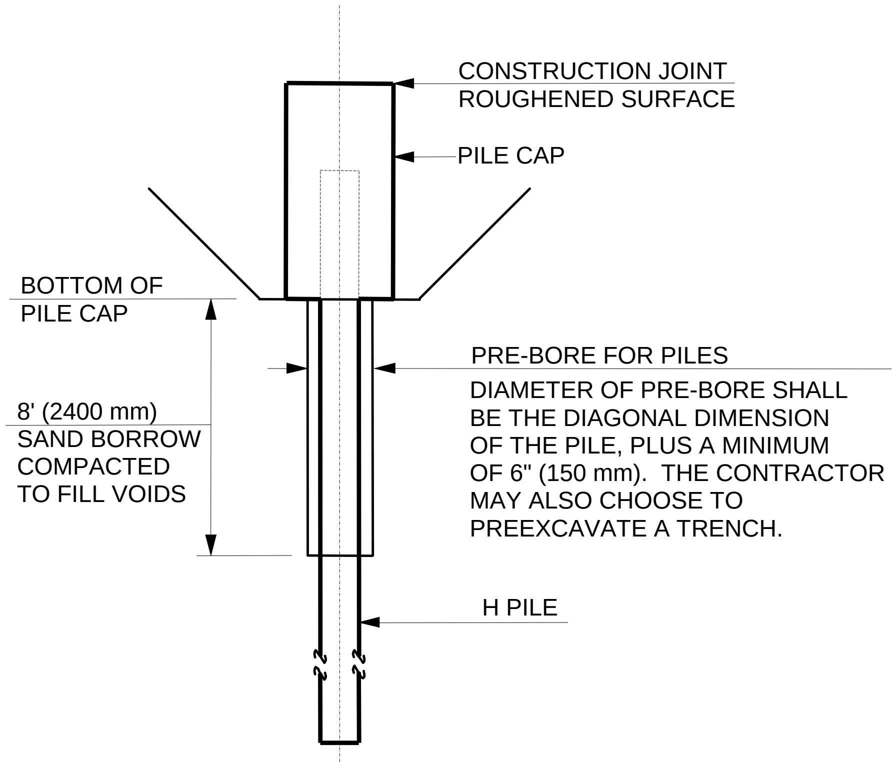

Figure 2.5-1 Pre-Bore Detail for Difficult Driving Conditions

SECTION 3 LOADS

3.1 GENERAL INFORMATION

This section describes the general approach to loading an Integral Abutment structure. This section may be used if the structure meets the criteria for the Simplified Design Method (Section 2.2). If the structure deviates from the criteria, a more detailed look at the loading stages of the structure will be necessary.

3.2 APPLICATION OF LOADS

Loads on an Integral Abutment bridge will be applied over several construction stages. Each stage will require different analysis models.

3.2.1 Construction Stage

Loads applied at this stage will be modeled as simply supported (see Figure 3.2.1-1.) The designer shall make every attempt to minimize moments to the pile cap due to p-delta effects when applying these loads.

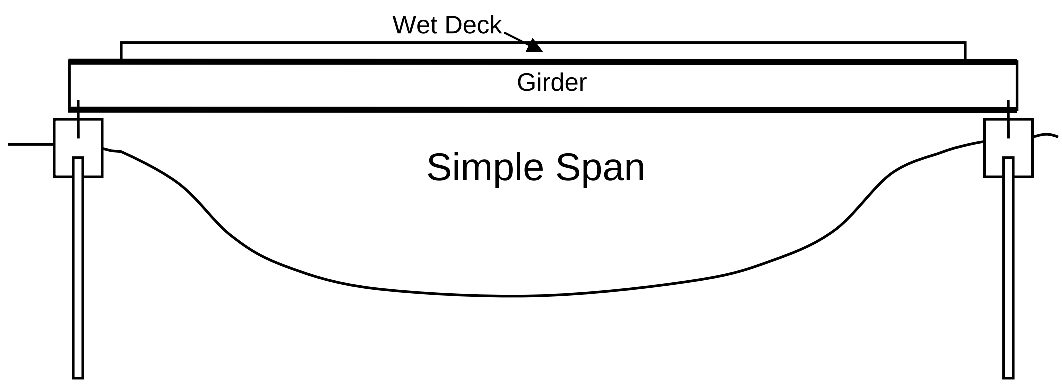

Figure 3.2.1-1 Simple Span Construction Stage

3.2.1.1 Permanent Dead Loads on Pile Cap

Dead loads include the construction bearing mechanisms, girders (or beams) self weight and placed concrete decking and backwalls. The loading effects will be applied to the non-composite girder section and will be additive to loads applied at future phases.

3.2.1.2 Construction Dead and Live Loads

Construction dead loads include removable concrete forms, screed rails, and other falsework. Construction live loads include the screed machine and other mobile loads on the deck during construction. The effects of these temporary loads will be applied to the girder and the temporary bearings for construction design checks.

3.2.1.3 Permanent Dead Load on Piles

Permanent dead loads on the pile cap translate through the pile cap to the piles and add to the load effects of the dead load of the pile cap. The designer may divide this load over the piles evenly, or may use the Strut-and-Tie method, suggested by LRFD Section 5.6.3.1, as an alternative method to obtain pile loads.

3.2.2 Final Stage

Loads applied at this phase induce a moment on the backwall/pile cap (see Figure 3.2.2-1.) This moment translates down into the piles. The moment effects from noncomposite and composite dead loads, live loads, temperature, shrinkage and creep will be additive. The loads applied at this phase translate to the piles through the pile cap.

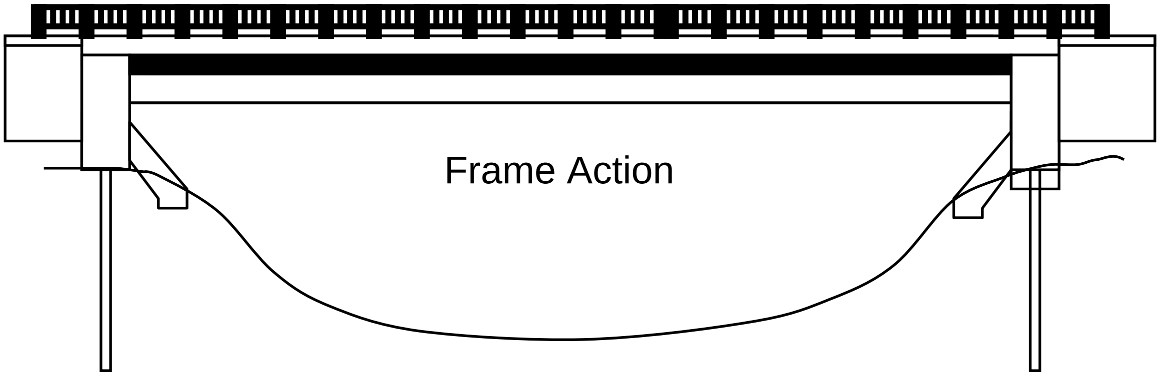

Figure 3.2.2-1 Final Rigid Frame Stage

3.2.2.1 Composite Permanent Dead Loads

Permanent dead loads applied to the composite section include: pavement, railing, lights, curbs and sidewalks. For the Simplified Design Method, these loads shall be applied in two ways. For the mid-span moments, treat the span as simply supported. No advantage of the frame action should be considered in the design. For the negative moment region at the end of the bridge, treat the span as a frame.

3.2.2.2 Live Loads

The AASHTO Design Live Load causes large end moments in the structure, especially in long structures. This load shall be applied similarly to permanent dead load (see section 3.2.2.1).

3.2.2.3 Longitudinal Effects

Thermal deformations, differential shrinkage, creep and breaking forces cause longitudinal load effects on the backwall and add to the end moments. Thermal deformation will cause a net longitudinal displacement on the pile cap. This deflection can be used in L-Pile when analyzing the pile capacity.

3.2.2.4 Earth Loads

Depending on the end of bridge details used, the effects of earth pressure may be significant. Unless elasticized EPS geofoam products are used, earth pressures will be applied directly to the backwall. This will create passive pressure opposing longitudinal expansion, and contribute to compressive action in the form of active earth pressure.

SECTION 4 STRUCTURAL ANALYSIS AND EVALUATION

4.1 GENERAL INFORMATION

The following is intended to provide the designer guidance in performing an integral abutment design. The reader is directed to the list of references at the end of the guideline for more detailed information regarding integral abutment design. In Appendix B of the guideline, a complete design example of the *Simplified Design Method* is provided for reference. Other references are available through members of the VTrans Integral Abutment Committee.

4.2 STRUCTURAL DESIGN CRITERIA

In addition to the requirements of Section 2.2.1 the following criteria shall be followed for the *Simplified Design Method*:

- The pile unbraced length shall be limited to 22 ft (*6.7 m*) or less.
- The non-composite dead load rotation at the end of girders shall be limited to 0.02 radians or less. (see section 6.2)

4.3 DESIGN METHODOLOGY

4.3.1 Simplified Design Method

The *Simplified Design Method* is provided to ease the design process of integral abutment bridges. To date, most integral abutment bridges built by the State of Vermont have complied with the *Simplified Design* criteria (see sections 2.2.1 and 4.2). The criteria have been developed from proven experience. From this experience, some general assumptions have been made such as:

- all loads are applied on a simply supported structure for superstructure load effects;
- all loads are applied on a frame for calculating the negative bridge end moments;
- skews 20° or less have no effect on the behavior of the structure;
- steel H-Piles will be made of 50 ksi (*345 MPa*) steel; and
- small dead and live load rotations have minimal effects on the structure.

A vital assumption in the *Simplified Design Method* is the plastic moment of the pile section is effected by the applied axial load. As the axial load on the pile increases, the moment ($M_{p'}$) which causes a plastic hinge will decrease. Once the plastic hinge is formed, the pile head could be considered a pin with a constant moment applied. (See Section 4.5.2.) All axial forces will transfer through the plastic hinge as determined by the interaction equation in LRFD Section 6.9.2.2 using $M_{p'}$ as a limiting condition for the applied moment.

Several details have been provided in the guideline that will aid in developing a set of plans for an integral abutment structure. For example, bearing schemes for prestressed concrete and steel structures are provided. These are proven details and may be used on integral abutment designs meeting the *Simplified Design Method*.

Bridges designed by this process may be slightly more conservative than from a more detailed design process. Structures that do not meet the criteria for the *Simplified Design Method* shall be designed with greater detail. The extent of the detailed design depends on how much of the criteria are met. If the bridge does not meet one criterion then the detailed design will only be related to that criterion. All other assumptions presented in this guide may be used without alteration.

4.3.2 Detailed Design

At the discretion of the designer, a more detailed design may be performed on any integral abutment structure. The designer must perform a detailed design for bridge elements that exceed the Simplified Design Method criterion. Detailed designs may include:

- construction bearing design (heavier loads or more significant rotations),
- calculating the effects of higher skews,
- torsion effects caused by curved girders,
- calculations for alternative pile choices,
- effects caused by longer bridge lengths or longer spans,
- effects of deep abutments, and
- effects caused by short pile embedment.

4.4 SUPERSTRUCTURE

Conventional deck and girder designs, similar to those performed for curtain wall and backwall bridges with bearings, remain appropriate for designing steel superstructures for integral abutment bridges. Conventional slab and prestressed member designs remain appropriate for designing concrete superstructures for integral abutment bridges.

4.4.1 Bridge End and Anchorage General Details

Use Figure 4.4.1-1 in selecting a bridge end detail for integral abutment bridges. **Please note that the Bridge End Details, types "A" through "I", can be found in the VTrans LRFD Structures Manual.** Refer to Sections 5 and 6 for details on concrete and steel bridge end details.

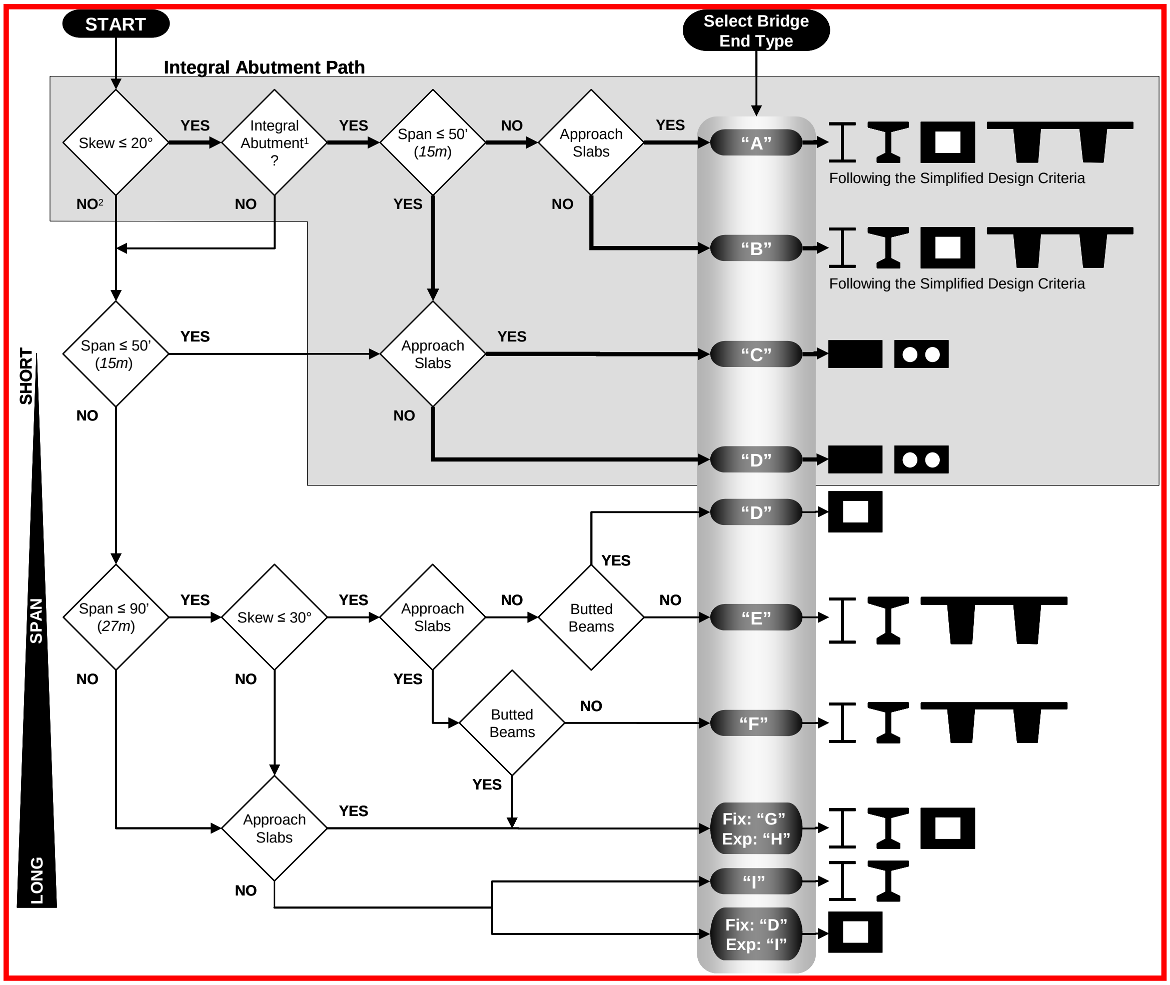

Figure 4.4.1-1 Bridge End Detail Selection Flowchart[1]

4.5 SUBSTRUCTURE

4.5.1 Abutment Movement

4.5.1.1 Thermal Movement

Movement in the abutment is in part caused by thermal expansion and contraction caused by fluctuations in the ambient temperature on the structure. The magnitude of this expansion and contraction tend to be less in concrete structures than in steel structures. Because of the heat retention capability of concrete, thermal variations in concrete bridges tend to be smaller than steel bridges. The following may be used for calculating thermal movement: (Refer to LRFD Section 3.12.2.1).

Determine the ambient temperature at which the girders will most likely be integrated with the abutments. This will aid with calculating the maximum possible thermal deflection. Construction typically occurs within the temperature range of 20°F to 70°F (*-6°C to 21°C*).

- Steel
  - o The AASHTO temperature range is 150°F (85°C.) or between -30°F to 120°F (-35°C to 50°C)
  - o The coefficient of thermal expansion is 0.0000065/°F (0.0000117/°C). The thermal movement rate is

[1] Changed July 15, 2009 - Modified chart from 2009 LRFD Structures Manual.

±1.17 inch /100 ft (99 mm / 100 m) of bridge length for this range.

  - Thermal movement is distributed equally to both abutments.
  - For the maximum bridge length of 395 ft (119 m), the absolute maximum movement of 4.62 inch (118 mm) divided over the two abutments yields 2.31 inch (59 mm) of thermal movement at each abutment. However, it is unlikely the beams will be set at either of the temperature extremes. Typically in Vermont, beams erected in the summer, would have a temperature lower then 100°F (38°C) and the temperature in the winter would typically be higher than 32°F (0°C). Reasonable maximum temperature decrease and increase ranges would therefore be 130°F (73°C) for thermal contraction and 88°F (50°C) for thermal expansion respectively. This would result in using an anticipated maximum displacement of 2.00 inch (51 mm) in the design of a 395 ft (119 m) bridge.

- Concrete
  - Temperature range is 80°F (45°C.) or between 0°F to 80°F (-18°C to 27°C)
  - The coefficient of thermal expansion is 0.0000060/°F (0.0000108/°C). The thermal movement rate is ±0.576 inch / 100 ft (49 mm / 100 m) of bridge length.
  - Thermal movement is distributed equally to both abutments.
  - For the maximum concrete bridge length of 695 ft (210 m), the absolute maximum movement of 4.00 inch (103 mm) divided over the two abutments yields 2.00 inch (51 mm) of thermal movement at each abutment. However, it is unlikely the beams will be set at the lower temperature extreme. If a concrete beams is placed in the summer, its temperature may be as high as 80°F (27°C) as suggested by LRFD Section 3.12.2.1, however based on winter construction practice in Vermont; beams will likely be placed in temperatures higher than 32°F (0°C). Reasonable maximum temperature decrease and increase ranges would therefore be 80°F (45°C) for thermal contraction and 48°F (27°C) for thermal expansion respectively.

4.5.1.2 Shrinkage and Creep

The abutment will be required to accommodate movement due to creep and shrinkage. This issue should be addressed mostly in designs of cast-in-place or prestressed concrete superstructures. The designer may refer to the 6th edition of the PCI Design Handbook, Section 4.7 for more information on how to calculate this displacement.

4.5.1.3 Total Allowable Movement

Total abutment movement shall be limited to 2 inches (*50 mm*) per abutment.

4.5.1.4 Grade of Steel

Steel piles shall be grade 50 (*345*) steel. (ASTM A572, VTrans 2006 Spec 730)

4.5.1.5 Pile Selection

Piles should be designed according to the requirements of LRFD Section 6.9 and the requirements of Section 10.4 of this guideline. Use one of the piles listed in Table **4.5.1.5-**1 for integral abutment bridges. For stability calculations K may be conservatively assumed to be 1.0. If the requirements of Section 2.2 are met, the Nominal Structural Pile Resistance (NSPR) can be calculated by,

$$P_n = 0.66^{\lambda} F_y A_s \geq R_n \qquad (4.5.1.5\text{-}1)$$

An initial pile selection can be approximated by the equation $P_n = CF_yA_s$ assuming C = 0.80 for weak axis bending and 0.92 for strong axis bending. This can be used for any Grade 50 pile with an approximate unbraced length of 15 ft (*4.6 m*). Once a more refined unbraced length has been determined, Table **4.5.1.5-**2 can be used to further refine the pile selection. Table **4.5.1.5-**3 contains axial load capacities of piles with unbraced lengths up to 28 ft (*8.4 m*) based in K=1.0. The values in Table **4.5.1.5-4** are based on K=1.2 and Table **4.5.1.5-5** are based on K=2.1. Based on the specific nature of the project, the content of the table may not be appropriate for final design. All HP Sections listed in the AISC Manual of Steel Construction, LRFD 3rd ed. and AISC Steel Construction Manual 13th ed. comply with the slenderness requirements of LRFD Section 6.9.4.2 except for the HP14x73 (*HP360x108*) and HP12x53 (*HP310x 79*) sections. (See Section 4.5.2.2). These two sections should be avoided.

Pile sections may be designed for primary bending without considering biaxial bending for skews up to 20°. For skews over 20°, biaxial bending should be considered in the pile design.

The Geotechnical Report should include a statement indicating the ability for the soil to provide lateral support for the pile. The designer will need to know if the soil conditions will allow the pile to be fully braced against buckling. In the absence of guidance in the Geotechnical Report, when the pile extends through layers of soft soil, the designer should assume that the pile acts like an unbraced column. See Figure 4.5.2.1-2 for the unbraced length defined. The corresponding AASHTO design requirements should be followed. In addition to the above, the Geotechnical Report will usually recommend a pile size as well.

Table 4.5.1.5-1 Pile Areas and Fully Braced Axial Strength[2]

Pile	A_s	F_yA_s[3]
HP 14x117 (*360x174*)	**34.4 in² (*22 194 mm²*)**	**1720 kips (*7651 kN*)**
HP 14x102 (*360x152*)	**30.0 in² (*19 355 mm²*)**	**1500 kips (*6672 kN*)**
HP 14x 89 (*360x132*)	26.1 in² (*16 800 mm²*)	1305 kips (*5805 kN*)
HP 12x 84 (*310x125*)	**24.6 in² (*15 871 mm²*)**	**1230 kips (*5471 kN*)**
HP 12x74 (*310x110*)	**21.8 in² (*14 064 mm²*)**	**1090 kips (*4849 kN*)**
HP 12x63 (*310x93*)	18.4 in² (*11 900 mm²*)	920 kips (*4092 kN*)
HP 10x57 (*250x85*)	**16.8 in² (*10 839 mm²*)**	**840 kips (*3737 kN*)**
HP 10x42 (*250x62*)	12.4 in² (*8 000 mm²*)	620 kips (*2758 kN*)
HP 8x36 (*200x53*)	10.6 in² (*6 840 mm²*)	530 kips (*2358 kN*)

Table 4.5.1.5-2 Preliminary Design Values for 'C'

Design axis	Unbraced length (lb)			Initial Design
	< 12 ft (*< 3.5m*)	12 ft to 16 ft (*3.5m to 5.0m*)	>16 ft (*> 5.0m*)	
Weak-axis	0.90	0.77	0.60	0.80
Strong-axis	0.96	0.92	0.82	0.92

Table 4.5.1.5-3 Pile Axial Capacity (US Units, P_n, *kips*, F_y*=50 ksi, K=1.0, See LRFD Section 6.9*)[2]

US K=1.0	Primary Bending in Weak Axis													
	Unbraced Length (*ft*)													
Pile[4]	2	4	6	8	10	12	14	16	18	20	22	24	26	28
14x117	1714	1697	1670	1633	1586	1530	1467	1397	1322	1243	1161	1078	994	910
14x102	1495	1480	1456	1422	1381	1332	1276	1214	1148	1078	1006	932	858	785
14x 89	1300	1287	1266	1236	1199	1156	1107	1052	944	933	869	804	740	676
12x 84	1224	1206	1177	1138	1089	1033	970	902	831	758	685	612	543	474
12x 74	1084	1068	1042	1007	964	913	857	796	732	667	602	537	475	414
12x 63	915	901	879	848	811	767	718	666	611	555	499	445	392	340
10x 57	834	816	788	751	705	653	597	537	477	418	361	306	260	224
10x 42	615	602	581	552	517	478	435	391	346	301	259	218	186	160
8x 36	524	507	480	444	402	356	309	262	217	176	145	122	104	89
US K=1.0	Primary Bending in Strong Axis													
	Unbraced Length (*ft*)													
Pile[4]	2	4	6	8	10	12	14	16	18	20	22	24	26	28
14x117	1717	1711	1701	1687	1670	1648	1623	1595	1158	1529	1491	1451	1409	1365
14x102	1498	1492	1483	1471	1455	1436	1414	1389	1006	1331	1298	1263	1226	1187
14x 89	1303	1298	1290	1279	1266	1249	1229	1207	872	1156	1127	1096	1063	1029
12x 84	1228	1222	1212	1199	1182	1161	1138	1111	743	1049	1015	979	941	901
12x 74	1088	1083	1074	1062	1047	1028	1007	983	655	928	898	865	831	796
12x 63	918	914	906	896	883	867	849	828	548	781	755	727	698	668
10x 57	837	831	822	808	791	770	747	720	439	661	628	595	560	525
10x 42	618	613	606	596	583	567	549	529	319	485	460	435	409	383
8x 36	528	522	512	499	483	463	442	418	210	365	338	310	283	256

Table 4.5.1.5-3 Pile Axial Capacity (Metric Units, P_n, *N*, *Fy=345 MPa*, *K=1.0*, *See LRFD Section 6.9*)[5]

MET K=1.0	Primary Bending in Weak Axis													
	Unbraced Length (*m*)													
Pile[6]	0.6	1.2	1.8	2.4	3.0	3.6	4.2	4.8	5.4	6.0	6.6	7.2	7.8	8.4
360x174	**7633**	**7561**	**7443**	**7281**	**7078**	**6837**	**6563**	**6261**	**5935**	**5591**	**5234**	**4869**	**4501**	**4134**
360x152	**6656**	**6593**	**6488**	**6344**	**6164**	**5951**	**5709**	**5442**	**5154**	**4850**	**4535**	**4213**	**3890**	**3568**
360x132	5790	5734	5642	5515	5356	5168	4954	4718	4464	4197	3920	3637	3353	3071
310x125	**5450**	**5374**	**5249**	**5080**	**4870**	**4625**	**4352**	**4056**	**3745**	**3426**	**3105**	**2788**	**2479**	**2184**
310x110	**4829**	**4761**	**4649**	**4497**	**4309**	**4089**	**3844**	**3580**	**3302**	**3017**	**2730**	**2447**	**2173**	**1912**
310x 93	4076	4016	3919	3788	3625	3435	3224	2996	2757	2513	2268	2027	1794	1564
250x 85	**3714**	**3640**	**3519**	**3356**	**3159**	**2933**	**2686**	**2427**	**2164**	**1904**	**1652**	**1406**	**1198**	**1033**
250x 62	2741	2684	2592	2468	2318	2147	1961	1766	1568	1374	1186	1004	856	738
200x 53	2334	2261	2144	1989	1808	1608	1400	1193	995	810	669	562	479	413
MET K=1.0	Primary Bending in Strong Axis													
	Unbraced Length (*m*)													
Pile[6]	0.6	1.2	1.8	2.4	3.0	3.6	4.2	4.8	5.4	6.0	6.6	7.2	7.8	8.4
360x174	**7648**	**7622**	**7579**	**7518**	**7441**	**7349**	**7240**	**7118**	**6981**	**6831**	**6670**	**6497**	**6314**	**6123**
360x152	**6670**	**6647**	**6608**	**6555**	**6487**	**6405**	**6310**	**6201**	**6080**	**5948**	**5806**	**5653**	**5492**	**5323**
360x132	5803	5782	5748	5701	5642	5569	5485	5390	5283	5167	5041	4907	4765	4617
310x125	**5467**	**5442**	**5400**	**5343**	**5269**	**5181**	**5079**	**4964**	**4836**	**4697**	**4548**	**4390**	**4225**	**4054**
310x110	**4845**	**4822**	**4785**	**4733**	**4668**	**4589**	**4497**	**4393**	**4279**	**4155**	**4022**	**3881**	**3733**	**3580**
310x 93	4089	4070	4038	3993	3937	3869	3790	3701	3603	3496	3382	3261	3134	3003
250x 85	**3731**	**3705**	**3662**	**3603**	**3529**	**3440**	**3338**	**3224**	**3099**	**2965**	**2824**	**2678**	**2527**	**2373**
250x 62	2753	2734	2702	2657	2601	2534	2457	2371	2277	2176	2070	1960	1847	1733
200x 53	2351	2326	2284	2228	2157	2073	1979	1875	1764	1648	1528	1407	1286	1167

Table 4.5.1.5-4 Pile Axial Capacity (US Units, P_n, *kips*, F_y*=50 ksi*, *K=1.2*, *See LRFD Section 6.9*)[5]

US K=1.2	Primary Bending in Weak Axis													
	Unbraced Length (*ft*)													
Pile[7]	2	4	6	8	10	12	14	16	18	20	22	24	26	28
14x117	**1712**	**1688**	**1649**	**1596**	**1530**	**1454**	**1368**	**1276**	**1178**	**1078**	**977**	**878**	**781**	**688**
14x102	**1493**	**1472**	**1437**	**1390**	**1332**	**1264**	**1188**	**1107**	**1021**	**933**	**844**	**757**	**672**	**591**
14x 89	1299	1280	1249	1208	1157	1097	1030	958	882	805	727	651	577	504
12x 84	**1221**	**1196**	**1155**	**1100**	**1033**	**957**	**874**	**788**	**700**	**613**	**530**	**448**	**382**	**329**
12x 74	**1082**	**1060**	**1023**	**974**	**914**	**845**	**771**	**694**	**615**	**538**	**464**	**392**	**334**	**288**
12x 63	913	894	862	819	767	708	645	578	511	445	382	322	274	236
10x 57	**832**	**807**	**767**	**715**	**654**	**585**	**514**	**442**	**373**	**306**	**253**	**213**	**181**	**156**
10x 42	614	595	565	525	478	427	373	319	268	219	181	152	129	112
8x 36	522	497	460	411	357	300	244	191	151	122	101	85	72	62

1.1.1.1

[2] Bold type indicate preferred pile choices.
[3] Grade 50 (*345*) Steel F_y = 50 ksi (*345 MPa*).
[4] The HP 14x73 and HP 12x53 have not been included in these tables because they do not comply with the Simplified Design Method.
[5] Bold type indicate preferred pile choices.
[6] The HP 360x108 and HP 310x79 have not been included in these tables because they do not comply with the Simplified Design Method.
[7] The HP 14x73 and HP 12x53 have not been included in these tables because they do not comply with the Simplified Design Method.

US K=1.2	Table 4.5.1.5-4 (*cont.*) Primary Bending in Strong Axis													
	Unbraced Length (*ft*)													
Pile[7]	2	4	6	8	10	12	14	16	18	20	22	24	26	28
14x117	**1717**	**1708**	**1694**	**1674**	**1649**	**1618**	**1583**	**1543**	**1499**	**1452**	**1401**	**1348**	**1292**	**1234**
14x102	**1497**	**1490**	**1477**	**1459**	**1437**	**1410**	**1379**	**1344**	**1305**	**1263**	**1218**	**1171**	**1122**	**1071**
14x 89	1303	1296	1285	1269	1249	1226	1198	1167	1133	1096	1057	1016	972	928
12x 84	**1227**	**1219**	**1205**	**1186**	**1162**	**1133**	**1100**	**1063**	**1023**	**979**	**934**	**886**	**837**	**787**
12x 74	**1087**	**1080**	**1068**	**1051**	**1029**	**1003**	**974**	**940**	**904**	**866**	**825**	**782**	**738**	**694**
12x 63	918	911	901	886	867	845	820	791	760	727	692	656	618	580
10x 57	**837**	**829**	**814**	**795**	**771**	**742**	**709**	**674**	**635**	**595**	**554**	**511**	**469**	**428**
10x 42	618	611	601	586	568	546	522	495	466	436	404	373	341	310
8x 36	527	519	505	487	464	437	408	377	344	311	278	246	215	185

Table 4.5.1.5-4 Pile Axial Capacity (Metric Units, P_n, *N*, *Fy=345 MPa*, *K=1.2*, *See LRFD Section 6.9*)[8]														
MET K=1.2	Primary Bending in Weak Axis													
	Unbraced Length (*m*)													
Pile[9]	0.6	1.2	1.8	2.4	3.0	3.6	4.2	4.8	5.4	6.0	6.6	7.2	7.8	8.4
360x174	**7622**	**7519**	**7351**	**7122**	**6837**	**6505**	**6133**	**5731**	**5306**	**4869**	**4427**	**3989**	**3562**	**3152**
360x152	**6647**	**6556**	**6406**	**6203**	**5951**	**5657**	**5329**	**4973**	**4599**	**4213**	**3825**	**3441**	**3067**	**2708**
360x132	5782	5702	5570	5390	5168	4908	4618	4305	3975	3637	3296	2960	2633	2320
310x125	**5439**	**5330**	**5153**	**4915**	**4625**	**4294**	**3933**	**3554**	**3169**	**2788**	**2419**	**2059**	**1755**	**1513**
310x110	**4819**	**4721**	**4562**	**4349**	**4089**	**3793**	**3470**	**3131**	**2787**	**2447**	**2120**	**1800**	**1534**	**1323**
310x 93	4067	3982	3844	3659	3435	3179	2901	2611	2316	2027	1748	1478	1260	1086
250x 85	**3703**	**3597**	**3426**	**3201**	**2933**	**2635**	**2322**	**2007**	**1701**	**1406**	**1162**	**977**	**832**	**718**
250x 62	2732	2651	2521	2350	2147	1922	1687	1451	1223	1004	830	698	594	512
200x 53	2323	2219	2055	1846	1608	1358	1112	878	694	562	465	390	333	287
MET K=1.2	Primary Bending in Strong Axis													
	Unbraced Length (*m*)													
Pile[9]	0.6	1.2	1.8	2.4	3.0	3.6	4.2	4.8	5.4	6.0	6.6	7.2	7.8	8.4
360x174	**7644**	**7607**	**7544**	**7458**	**7349**	**7217**	**7065**	**6893**	**6703**	**6497**	**6276**	**6044**	**5801**	**5549**
360x152	**6666**	**6633**	**6578**	**6502**	**6405**	**6289**	**6154**	**6002**	**5835**	**5653**	**5459**	**5254**	**5040**	**4818**
360x132	5800	5770	5722	5655	5569	5467	5348	5215	5067	4907	4736	4556	4368	4173
310x125	**5463**	**5427**	**5368**	**5285**	**5181**	**5057**	**4914**	**4754**	**4578**	**4390**	**4191**	**3984**	**3770**	**3551**
310x110	**4841**	**4809**	**4756**	**4682**	**4589**	**4477**	**4349**	**4206**	**4049**	**3881**	**3703**	**3517**	**3326**	**3131**
310x 93	4086	4058	4012	3949	3869	3773	3663	3540	3405	3261	3108	2950	2786	2620
250x 85	**3727**	**3690**	**3629**	**3545**	**3440**	**3316**	**3175**	**3020**	**2853**	**2678**	**2496**	**2312**	**2127**	**1943**
250x 62	2751	2722	2676	2613	2534	2440	2334	2217	2092	1960	1824	1686	1548	1412
200x 53	2347	2311	2252	2172	2073	1959	1831	1695	1552	1407	1262	1121	985	852

[8] Bold type indicate preferred pile choices.
[9] The HP 360x108 and HP 310x79 have not been included in these tables because they do not comply with the Simplified Design Method.

Table 4.5.1.5-5 Pile Axial Capacity (US Units, P_n, *kips*, F_y*=50 ksi*, *K=2.1*, *See LRFD Section 6.9*) [10]

US K=2.1	Primary Bending in Weak Axis													
	Unbraced Length (*ft*)													
Pile[11]	2	4	6	8	10	12	14	16	18	20	22	24	26	28
14x117	**1696**	**1624**	**1512**	**1368**	**1203**	**1028**	**853**	**688**	**543**	**440**	**363**	**305**	**260**	**224**
14x102	**1478**	**1415**	**1316**	**1188**	**1043**	**888**	**735**	**591**	**465**	**377**	**312**	**262**	**223**	**192**
14x 89	1286	1230	1142	1030	901	766	632	504	398	322	267	224	191	165
12x 84	**1204**	**1129**	**1015**	**874**	**722**	**571**	**430**	**329**	**260**	**211**	**174**	**146**	**125**	**108**
12x 74	**1067**	**1000**	**897**	**771**	**635**	**500**	**376**	**288**	**228**	**184**	**152**	**128**	**109**	**94**
12x 63	900	842	753	645	528	413	309	236	187	151	125	105	90	77
10x 57	**815**	**743**	**637**	**514**	**390**	**278**	**204**	**156**	**123**	**100**	**83**	**69**	**59**	**51**
10x 42	601	546	466	373	280	198	146	112	88	71	59	50	42	36
8x 36	505	437	343	244	160	111	82	62	49	40	33	28	24	20
US K=2.1	Primary Bending in Strong Axis													
	Unbraced Length (*ft*)													
Pile[11]	2	4	6	8	10	12	14	16	18	20	22	24	26	28
14x117	**1711**	**1685**	**1641**	**1583**	**1511**	**1427**	**1334**	**1234**	**1130**	**1024**	**918**	**815**	**715**	**618**
14x102	**1492**	**1469**	**1431**	**1379**	**1315**	**1241**	**1159**	**1071**	**980**	**886**	**794**	**703**	**617**	**532**
14x 89	1298	1277	1244	1198	1142	1077	1005	928	847	766	684	605	530	457
12x 84	**1221**	**1196**	**1155**	**1100**	**1033**	**957**	**874**	**787**	**699**	**612**	**529**	**448**	**381**	**329**
12x 74	**1082**	**1060**	**1023**	**974**	**914**	**845**	**771**	**694**	**615**	**538**	**464**	**392**	**334**	**288**
12x 63	913	894	862	820	768	710	646	580	513	448	385	324	276	238
10x 57	**831**	**805**	**764**	**709**	**645**	**575**	**501**	**428**	**357**	**291**	**241**	**202**	**172**	**148**
10x 42	613	594	563	522	473	420	365	310	258	210	173	146	124	107
8x 36	521	496	458	408	352	294	238	185	146	119	98	82	70	61

Table 4.5.1.5-5 Pile Axial Capacity (Metric Units, P_n, *N*, *Fy=345 MPa*, *K=2.1*, *See LRFD Section 6.9*) [10]

MET K=2.1	Primary Bending in Weak Axis													
	Unbraced Length (*m*)													
Pile[12]	0.6	1.2	1.8	2.4	3.0	3.6	4.2	4.8	5.4	6.0	6.6	7.2	7.8	8.4
360x174	**7551**	**7244**	**6758**	**6133**	**5414**	**4648**	**3881**	**3152**	**2493**	**2019**	**1669**	**1402**	**1195**	**1030**
360x152	**6584**	**6311**	**5882**	**5329**	**4694**	**4019**	**3346**	**2708**	**2138**	**1732**	**1431**	**1202**	**1025**	**883**
360x132	5727	5485	5106	4618	4059	3467	2877	2320	1829	1481	1224	1029	876	756
310x125	**5363**	**5041**	**4546**	**3933**	**3265**	**2601**	**1976**	**1513**	**1196**	**968**	**800**	**672**	**573**	**494**
310x110	**4752**	**4462**	**4018**	**3470**	**2873**	**2282**	**1728**	**1323**	**1045**	**847**	**700**	**588**	**501**	**432**
310x 93	4008	3757	3374	2901	2390	1886	1418	1086	858	695	574	483	411	355
250x 85	**3630**	**3320**	**2860**	**2322**	**1776**	**1276**	**937**	**718**	**567**	**459**	**380**	**319**	**272**	**234**
250x 62	2676	2440	2092	1687	1279	911	669	512	405	328	271	228	194	167
200x 53	2251	1955	1546	1112	734	510	375	287	227	184	152	127	109	94

[10] Bold type indicate preferred pile choices.
[11] The HP 14x73 and HP 12x53 have not been included in these tables because they do not comply with the Simplified Design Method.
[12] The HP 360x108 and HP 310x79 have not been included in these tables because they do not comply with the Simplified Design Method.

MET K=2.1	Table 4.5.1.5-5 (*cont.*) Primary Bending in Strong Axis													
	Unbraced Length (*m*)													
Pile[12]	0.6	1.2	1.8	2.4	3.0	3.6	4.2	4.8	5.4	6.0	6.6	7.2	7.8	8.4
360x174	**7618**	**7504**	**7318**	**7065**	**6752**	**6388**	**5984**	**5549**	**5094**	**4630**	**4165**	**3710**	**3272**	**2839**
360x152	**6643**	**6543**	**6378**	**6154**	**5878**	**5558**	**5201**	**4818**	**4418**	**4010**	**3603**	**3204**	**2821**	**2443**
360x132	5779	5690	5545	5348	5105	4823	4510	4173	3822	3465	3108	2760	2425	2097
310x125	**5439**	**5329**	**5152**	**4914**	**4624**	**4292**	**3931**	**3551**	**3166**	**2784**	**2415**	**2055**	**1751**	**1510**
310x110	**4819**	**4721**	**4562**	**4349**	**4089**	**3793**	**3470**	**3131**	**2787**	**2447**	**2120**	**1800**	**1534**	**1323**
310x 93	4067	3983	3846	3663	3440	3185	2909	2620	2327	2038	1760	1490	1270	1095
250x 85	**3701**	**3589**	**3410**	**3175**	**2896**	**2588**	**2265**	**1943**	**1633**	**1337**	**1105**	**928**	**791**	**682**
250x 62	2731	2647	2512	2334	2124	1893	1652	1412	1181	963	796	669	570	491
200x 53	2322	2215	2046	1831	1588	1334	1086	852	673	545	450	378	322	278

4.5.1.6 Pile Orientation

For bridges that meet the criteria in Section 2.2, piles should be oriented for weak axis bending with the webs of the piles detailed parallel to the centerline of bearing as shown in Figure 4.5.1.6-1. Bridges with a skew exceeding the limits in Section 2.2 may require the webs of the piles oriented perpendicular to the centerline of the bridge.

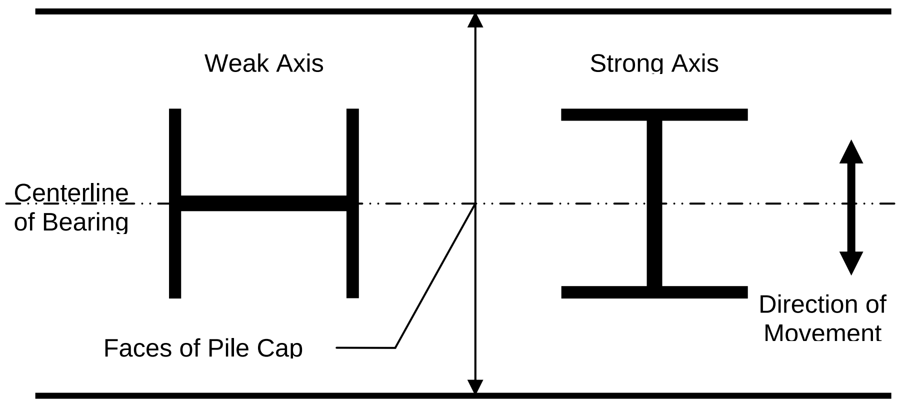

Figure 4.5.1.6-1 Typical orientation of piles in pile cap.

4.5.2 Pile Design

Referring to LRFD Section 6.15.2 commentary, the pile can be divided into two primary zones. The upper zone is subjected to flexure and axial loads. The lower zone is only subjected to axial loads as the flexural loads have been fully resisted in the upper zone. The pile is fully braced by the surrounding soil. Where the upper zone experiences very little damage due to driving, the lower zone may become damaged to some extent, especially if driven to bedrock.

In the upper zone, a plastic hinge may form at the point the pile enters the pile cap. This is a result of the pile cap displacing horizontally or rotation due to thermal, dead or live loads or various combinations of the three. A plastic hinge is an acceptable boundary condition for column design. To design a pile with a plastic hinge, assume the pile is pinned with a constant moment at the pile cap and pinned at the point of the first zero (0) moment. This is the first pile segment to analyze. The second segment will be between the top two zero (0) moments. The segment with the controlling axial strength will be used in determining the pile size to use in the design. Refer to the AISI, Highway Structures Design Handbook, Integral Abutments for Steel Bridges for more information. Refer to **Figure 4.5.2-1** .

The pile's axial capacity is determined by the controlling condition of the following (assuming uniaxial bending):

With L-Pile, apply the lateral deflection (δ), determined by the thermal displacement, shrinkage and creep; the axial load (P_u) applied to the pile and a fixed zero rotation of the pile cap.

The preferred ratio of the applied axial load (P_u) to the calculated compressive structural pile resistance (SPR)

(P_r) shall be greater then 0.2; otherwise the pile will be unnecessarily large.

$$P_u < P_r = \phi_c P_n \tag{4.5.2-1}$$

$$\frac{P_u}{P_r} \geq 0.2 \tag{4.5.2-2}$$

1. **Top Segment:** Enter the interaction equation in LRFD Section 6.9.2.2 (equation 1 below) with the applied axial load (P_u), the calculated SPR (P_r) and flexural (M_r) strengths based on the unbraced length of the top segment of the pile and equate to one. Solve for the applied moment. An axial load on a column will reduce the magnitude of the plastic moment (M_p). This resulting moment ($M_{p'}$) will be what creates a plastic hinge at the pile head. The applied moment will not be able to exceed this limit. For this portion of the pile, the axial resistance will be reduced by ϕ_c = 0.70 and the flexure resistance will be factored with ϕ_f = 1.00.

$$\frac{P_u}{P_r} + \frac{8}{9}\left(\frac{M_u}{M_r}\right) = 1.0 \tag{4.5.2-3}$$

Solve for M_u and equate to $M_{p'}$

$$M_u = M_{p'} = \frac{9.0}{8.0}\left(1.0 - \frac{P_u}{P_r}\right)M_r \tag{4.5.2-4}$$

2. If the applied moment (M_u) is greater then $M_{p'}$ a plastic hinge will form. A new analysis will need to be done using the lateral deflection and the axial load from step 1 and $M_{p'}$ calculated in step 2. This analysis will be used to revise the unbraced lengths of the pile. Recalculate the pile axial capacity in both the top segment and the second segment using the revised unbraced lengths. If M_u remains below $M_{p'}$ then the pile is still in the elastic range – no additional analysis is required.

3. **Second Segment:** From either the initial analysis or revised analysis obtain the maximum moment in the second segment. With this moment, the unbraced length of the second segment and the same axial load applied in step 1, ensure that LRFD Section 6.9.2.2 is satisfied for this segment. Use the same resistance factors as in step 1.

Lower Zone: For the lower zone of the pile, only calculate the axial capacity. The axial resistance is based on an unbraced length of zero and will be reduced by ϕ_c = 0.50 if the use of a pile tip is required (i.e. driving to bedrock) or ϕ_c = 0.60 if a pile tip is not required.

The controlling SPR will be the lowest axial capacity (P_r) of the top segment or the second segment of the upper zone or the lower zone of the pile. The SPR will be compared with the applied axial load.

The controlling Nominal Structural Pile Resistance (NSPR), will be the lowest nominal axial capacity (P_n) of the top segment or the second segment of the upper zone or the lower zone of the pile. The NSPR will be required for the drivability check (See Section 10.4.1).

Check the pile for flange and web buckling.

When M_u exceeds $M_{p'}$ at the pile head, the pile will develop a plastic hinge. The hinge allows the pile head to rotate with a constant moment ($M_{p'}$). The pile head transforms from a fixed connection to a pinned connection, thereby changing the effective length of the top segment for stability checks. The capacity of the top segment of the pile is enhanced if the plastic moment forms as described.

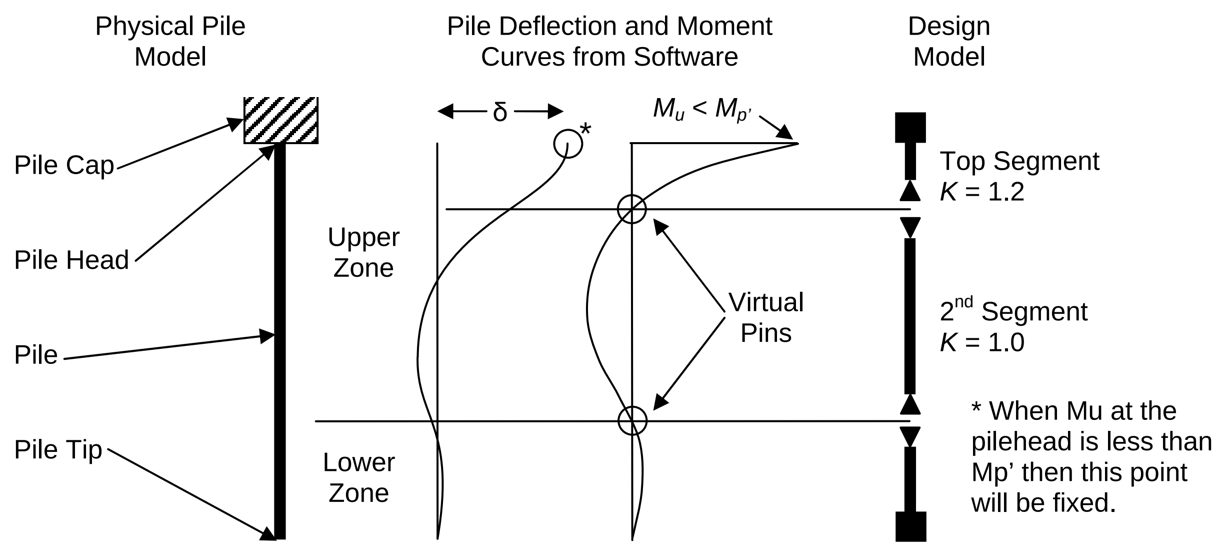

Figure 4.5.2-1 Pile Design Model when $M_u<M_{p'}$.

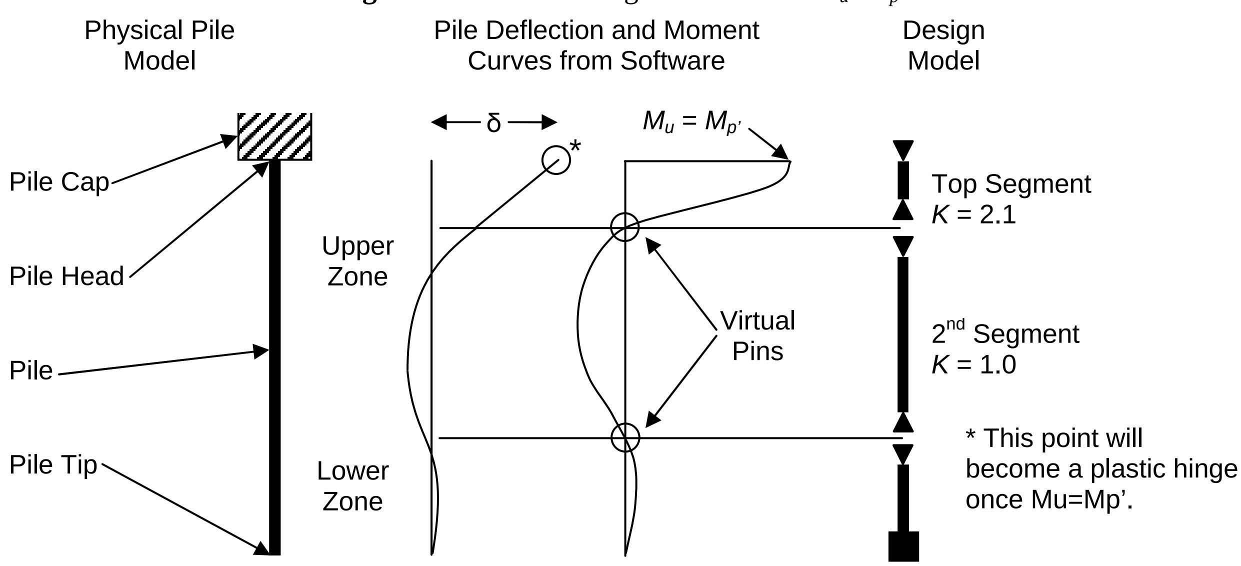

Figure 4.5.2-2 Pile Design Model when $M_u=M_{p'}$.

4.5.2.1 L-Pile® Software Analysis

The L-Pile® design software is available to the designer. This software is used to analyze pile-soil interaction in determining the moment developed throughout the pile due to the thermal movement demand of the structure. L-Pile® allows loads and deflections (lateral and rotational) to be included in a single load case for analysis. Typically, the Geotechnical Engineer will have developed an L-Pile® input file as part of the foundation analysis. Once the file is set up, the designer can use the file to make minor modifications, as necessary, to the pile geometry, orientation and soil profile to analyze the scour condition. The important elements of the design that are drawn from this output are the following:

4.5.2.1.1 Lateral Load at Pile Head

This is the lateral load required to generate the design thermal movement demand, or deflection, due to temperature loading.

4.5.2.1.2 Pile Deflection and Moment

The deflection and moment throughout the pile due to temperature loading can be output both in graphical (**Figure 4.5.2.1-1** and Figure 4.5.2.1-2) and tabular (Figure 4.5.2.1-3) formats. Included in the tabular output are the maximum negative moment at the pile head (X = 0 in.) and the maximum positive moment within the pile unbraced length. The location of the maximum positive moment is dependent upon the soil conditions and will vary from project to project.

4.5.2.1.3 Unbraced Lengths

The lengths along the pile:

- between the top of the pile (plastic hinge) and the first point of zero moment, or
- between the first and second points of zero moments (see **Figure 4.5.2.1-2** and **Figure 4.5.2.1-3**). This is the length used to calculate buckling stresses in the pile.

4.5.2.1.4 Depth to Fixity

Depth to fixity is the effective pile length or the depth to zero deflection in the pile. (See **Figure 4.5.2.1-1**). The effective pile length should be between the bottom of the pile cap to the depth of fixity.

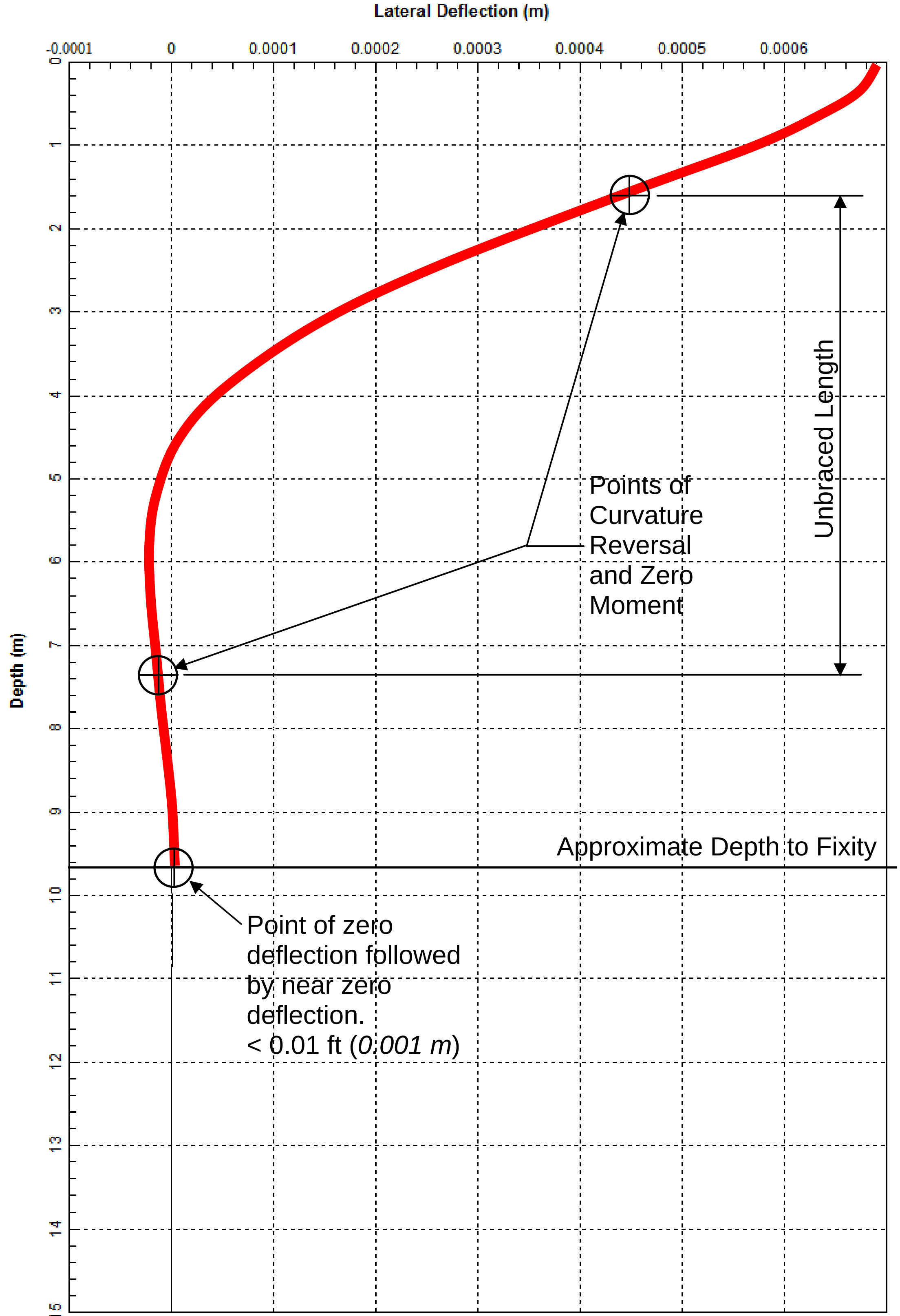

Figure 4.5.2.1-1 Graphical output of L-Pile run showing depth to fixity, shown in metric.

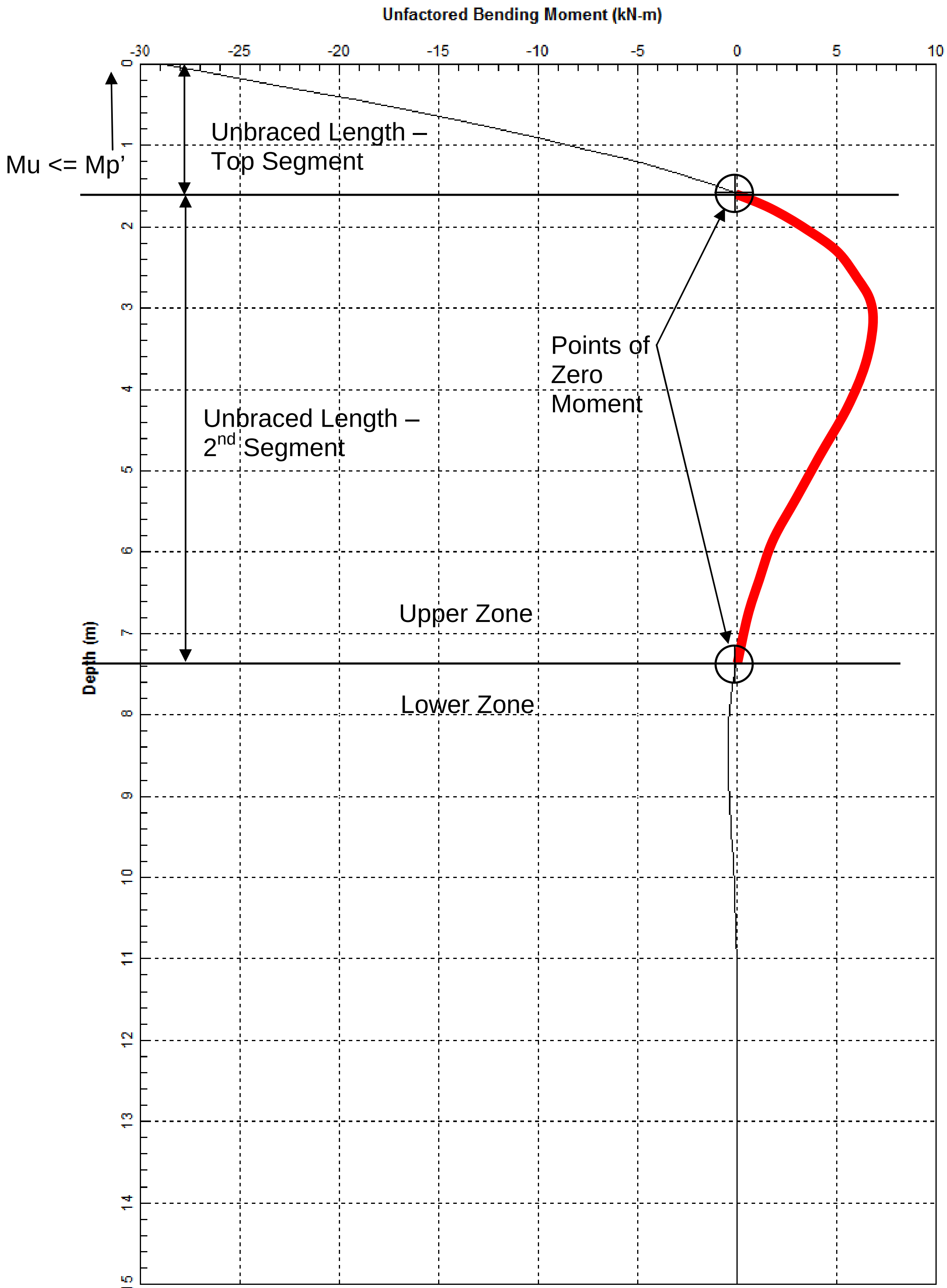

Figure 4.5.2.1-2 Graphical output of L-Pile® run showing the length of unbraced length, shown in metric.

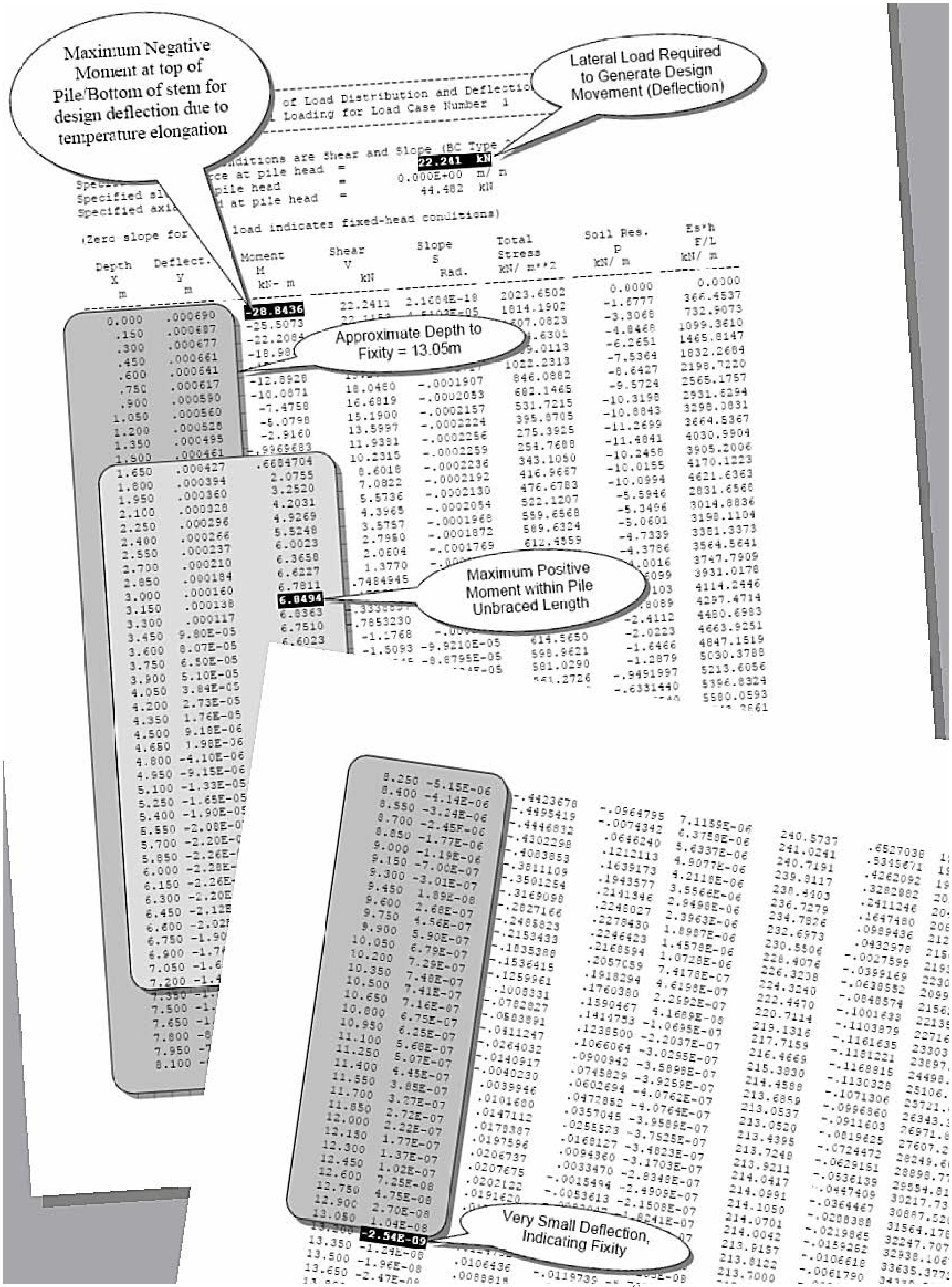

Figure 4.5.2.1-3 Typical printout of L-Pile® output.

4.5.2.2 Combined Axial Compression and Flexure

For both the non-scour and scour conditions, the unbraced pile length indicated in the L-Pile output should be analyzed as a beam-column subjected to combined axial load and bending using the requirements of LRFD Section 6.9.2.2. The applicable Group loadings from LRFD Table 3.4.1-1 should be used in the analysis. Typically, the final hydraulics report will indicate both the calculated depth of contraction scour and the elevation above which the piles should be designed to be freestanding.

Piles meeting the requirements of the applicable column or beam/column analysis are considered to be acceptable for use on the project.

4.5.3 Pile Cap

In designing the pile cap, the largest force that will have an effect on the design is the passive earth pressure of the backfill material placed behind the abutment. There will also be a moment induced by the live load and superimposed dead load rotations. The pile cap should be designed to resist the shear from the passive earth pressure, and the combined moment from passive earth pressure and live load and superimposed dead load rotation. The horizontal steel should be designed to resist the passive earth pressure, assuming the cap will act as a continuous beam between the girders.

The pile cap should be designed to resist:

- the horizontal from the passive earth pressure,
- the vertical shear from dead loads, and
- the horizontal and vertical combined moment from passive earth pressure, live load and superimposed dead load rotation.

Additional steel in the deck at each end may need to be designed to resist the above.

4.5.4 Wingwall Design

For monolithic wingwalls, design the horizontal steel at the intersection of the wingwall and the abutment to resist the cantilever forces induced by earth pressures acting behind the wingwall. Design wingwalls of the U-wall configuration for active earth pressure. If using in-line wingwalls, the earth pressure is passive due to the expansion force in the girders. With flared wingwalls, forces comprised of passive pressure acting in a direction perpendicular to the abutment centerline and an active component acting in a direction perpendicular to the centerline of the roadway are present. Do not put piles or a footing under wingwalls that are monolithically attached to the abutment. Wingwalls requiring a length longer then 10 ft (*3000 mm*) should be split into two segments. The first segment should be monolithically attached to the abutment and the second should be designed as a freestanding cantilever wall and should be isolated from the movement of the bridge.

4.6 PROJECT NOTES AND SPECIAL PROVISIONS

Project notes and general special provisions shall be written and included in a given project to explain unusual construction requirements. Both project notes and special provisions have been developed for several integral abutment bridge projects. The designer is encouraged to use these projects as examples. The intention is to move away from including excessive notes and special provisions by creating new pay items with corresponding specifications that relate to the construction of integral abutment bridges. These pay items have been created and assigned numbers. The specification for pile preparation is found in section 503 and the corresponding pay item number is 503.20. The specification for the steel piling for integral abutments is included in section 505 with the corresponding pay item numbers.

4.7 LOAD RATING

Integral abutment bridges should be load rated without any beneficial contribution of negative moment regions at the abutments. This conservative approach will save time developing a load rating. If any other assumptions are made when load rating these structures they should be carefully documented on the plans for future load rating purposes.

SECTION 5 CONCRETE STRUCTURES

5.1 GENERAL INFORMATION

Refer to the Structures Manual for more information.

5.2 PRESTRESS SUPERSTRUCTURE SPECIFIC DETAILS

Prestressed concrete superstructures should be designed with fixed end supports. However, there are circumstances that justify using simply supported end details typically used for conventional bridge configurations. It may be advantageous to design shorter decks using deck beams, simply supported. Longer spans, regardless of the beam or girder used, will benefit from fixed ends.

5.2.1 Voided Slab and Box Beam Bridge Decks

The designer may either choose to design Multi-beam bridge decks as integral with the substructure or as simply supported. Considering thermal movement is not significant for spans which utilizes multi-beam decks, either method will not significantly change the design of the substructure. To ease design requirements, multi-beam bridge decks need not be integral with the abutments. End details for Voided Slabs and Box Beams need not differ from those published in the Structures Manual. Design the deck as a conventional simple span. Live load and superimposed dead load negative moments effects at the abutment need not be considered.

Piles for non integrated decks shall be oriented with the strong bending axis in line with the controlling lateral load caused by the stream, ice, earth pressures or other lateral loads.

If the Voided Slabs or Box Beams will be designed to be integral with the substructure, a continuity connection will need to be designed and detailed to account for the rigid frame action of the composite dead load and live load. See Figure 5.2.1-1. This continuity is provided by extending some of the prestress strands and bending them 90° as shown in the detail. The number of strands to be extended and bent depends on the design. A quick guide is available from PCI-Northeast website (http://pcine.org/resources) titled "*Prestressed Concrete Girder Continuity Connection*" (PCINER-98-PCGCC). This guide is written for working stress and utilizes an ultimate strength check; however it may be used to estimate the number of prestressing strands required for the continuity connection at the end of the deck. For deck beam, the use of a timber block as a temporary bearing may be appropriate to lower the cost of construction.

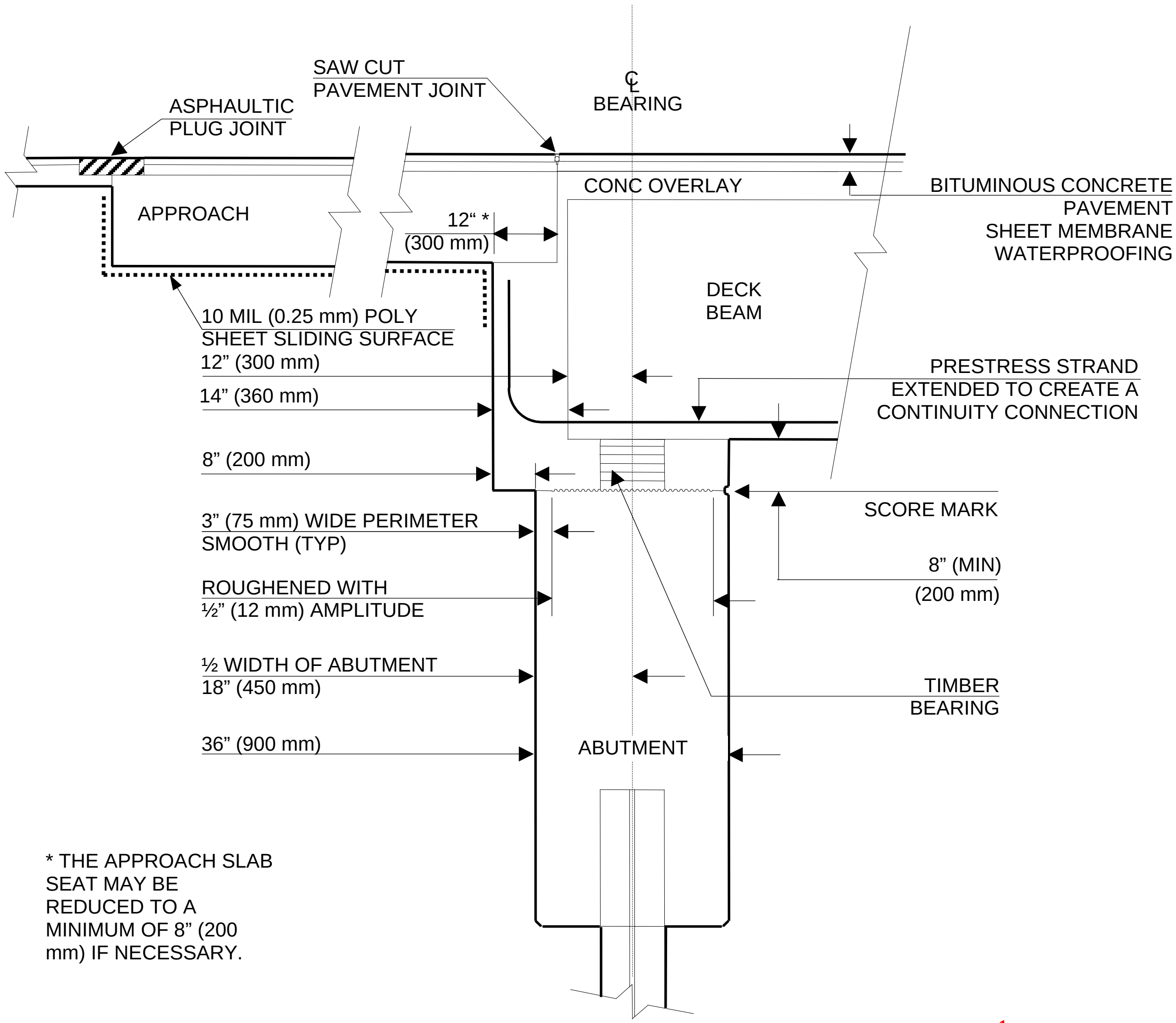

Figure 5.2.1-1 Bridge End Detail Type **A** (Prestressed Butted Deck Beam).[1]

5.2.2 Northeast Bulb-T (NEBT)

For details of the Bearing assembly, see Figure 5.2.2-1. The beam bearing assembly should include:

- a concrete pedestal support 8 inches (*200 mm*) high providing support for the full width of the section, and
- a 1 inch (*25 mm*) thick elastomeric pad with the same dimensions as the concrete pedestal.

Consider providing sleeves in the girder web for reinforcing steel according to design for the end of girder details. Refer to Figure 5.2.2-1 and Figure 5.2.2-2 for more information.

The following construction considerations should be specified in the plans:

- The pedestal form shall be checked for elevation prior to and after concrete placement. The surface of the pedestal shall be checked for level.
- Prior to placing the girders, check the elevation of the pedestals; grind and shim where necessary, then place the elastomeric pad on the pedestal.

[1] **Changed July 15, 2009 – Bridge End Detail "J" has been changed to "A" in the 2009 LRFD Structures Manual.**

- Spans over 90 ft (*27 000 mm*) may require some provisions to provide nominal expansion during construction.
- The construction joint between the pile cap and the curtain wall should include the following considerations:
  - o The cross slope of the pile cap's top surface shall match the cross slope of the bridge surface. (See **Figure 6.2-1**).
  - o The top surface of the pile cap shall be float finished to grade.
  - o The interior of the surface shall be intentionally roughed by raking parallel with the face of the abutment to an amplitude of ½ inch (*10 mm*)
  - o a width of no more than 3 inches (*75 mm*) around the perimeter of the surface shall remain smooth.
  - o detail a score mark on front face of the pile cap along the construction joint
  - o detail a minimum of 8 inches (*200 mm*) from theoretical seat elevation (*bottom of girder flange*) to the joint elevation

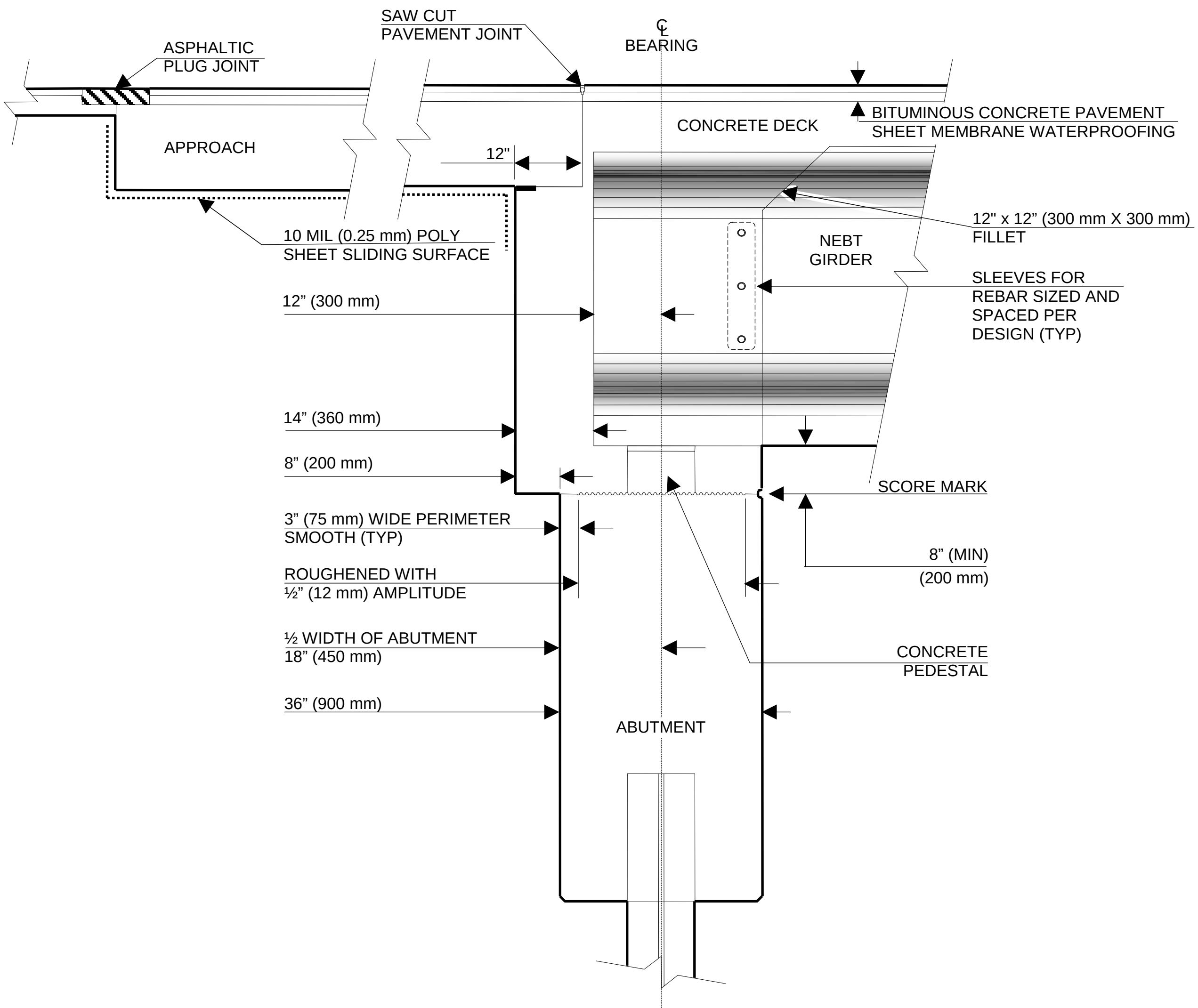

Figure 5.2.2-1 Bridge End Detail Type A (Prestressed Concrete NEBT Girder)[2,3]

[2] Decks constructed of AASHTO I-sections or spread box beam sections would have similar details.
[3] **Changed February 25, 2009 – Bridge End Detail "J" has been changed to "A" in the 2009 LRFD Structures Manual.**

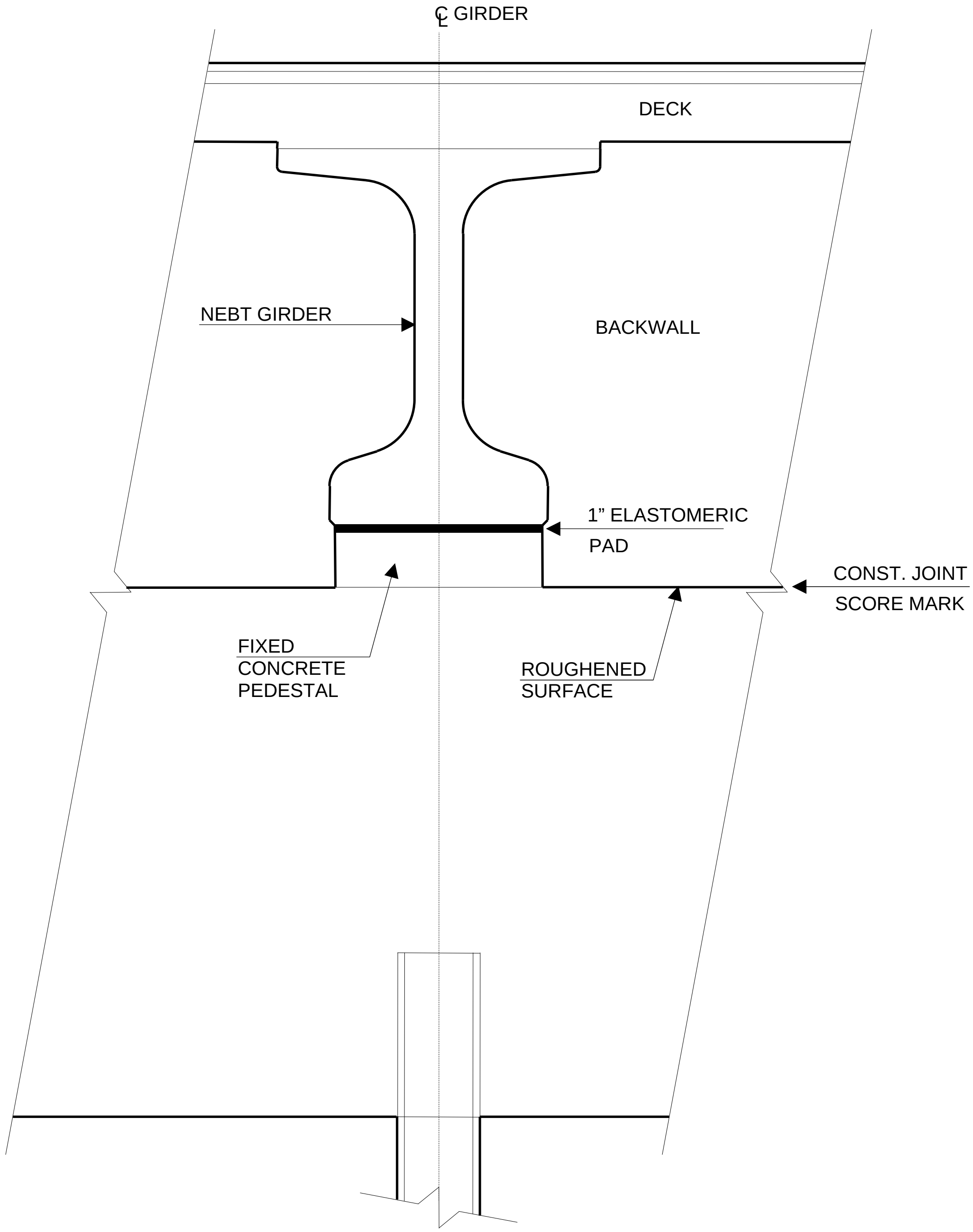

Figure 5.2.2-2 Bearing Assembly Cross Section (NEBT Girder)

5.2.2.1 Cast-In-Place Concrete Slab Decks

Cast-in-Place Concrete bridge decks are similar to prestressed deck beams. Refer to Section 5.2.1 for more information.

5.2.3 Design for Frame Action (Negative Moment) at Ends of Deck

Though the deck is assumed to behave like a simply supported deck and designed accordingly, because the ends are locked in, the designer shall provide negative moment reinforcement at the end of the deck. End moments are calculated using superimposed dead loads and live loads on a frame. Reinforcement will run from the deck down into the backwall of each abutment. See Figure 5.2.3-1.

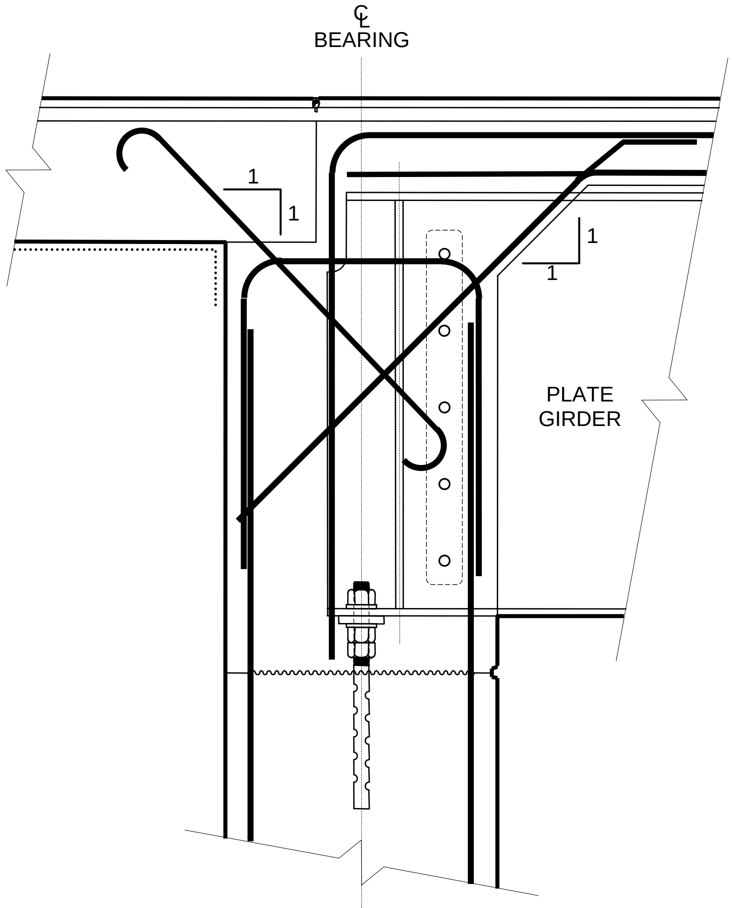

Figure 5.2.3-1 Negative Moment Reinforcement

SECTION 6 STEEL STRUCTURES

6.1 GENERAL INFORMATION

Refer to the Structures Manual for more information.

6.2 STEEL GIRDER SPECIFIC DETAILS

The bridge end detail should contain the following, as illustrated in Figure 6.2-2:

- Approach slabs should be designed to move with the pile cap.
- Use a 12 inch x 12 inch (*300 mm x 300 mm*) fillet in the deck.
- Locate the transverse deck joint according to design.
- The construction joint between the pile cap and the curtain wall should include the following considerations:
    - The cross slope of the pile cap's top surface shall match the cross slope of the bridge surface. (See **Figure 6.2-1**).
    - The top surface of the pile cap shall be float finished to grade.
    - The interior of the surface shall be intentionally roughed by raking parallel with the face of the abutment to an amplitude of ½ inch (*10 mm*).
    - A width of no more than 3 inches (*75 mm*) around the perimeter of the surface shall remain smooth.
    - Detail a score mark on front face of the pile cap along the construction joint.
    - Detail a minimum of 8 inches (*200 mm*) from theoretical seat elevation (*bottom of girder flange*) to the joint elevation.

For details of the Anchorage/Bearing assembly, see Figure 6.2-3. The beam anchorage should include:

- an assembly on each side of the web, securing the flanges of the girder including a nut and washer;
- grade 50 (*345*) or higher strength (A449, *F1554 or F568M*, *Class 8.8*) swedged and threaded anchor bolts on either side of the girder, 2 inches (*51 mm*) in diameter (minimum) **with the top threaded to within** 1 inch (*25 mm*) of the construction joint and **the bottom** embedded a minimum of 18 inches (*450 mm*);
- a leveling plate (*sole plate*), spanning the anchor bolts, fully supporting the full width of the bottom flange with the length and thickness determined by design (*1 inch or 25 mm minimum thickness for plate*);
- a steel plate washer on each side of the web, supporting the leveling plate;
- a set of double nuts, supporting the plate washers, on each side of the web; and
- a clearance of 8 inches (*200 mm*) at centerline of girder between the construction joint and the girder to allow for each item described above.

All steel components used in the Anchorage/Bearing assembly shall be uncoated steel matching the specification used for the beams except as noted above. The project plans or special provisions shall include a note to modify the bearing specifications appropriately.[1]

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[1] **Added on June 12, 2008. This text is to clarify the requirement to galvanize the leveling plate as suggested by the specifications for bearings. There is no need to galvanize the components of the Anchorage/Bearing assembly. Justification for this includes: this guideline considers the assembly a temporary bearing device and does not rely on the assembly for support once embedded in concrete; using the details in this guideline will protect the assembly from**

For situations where longitudinal slopes are over 5% or dead load rotations are over 0.02 radians, the designer may need to bevel the leveling plate. This bevel will need to accommodate the girder slope or the anticipated rotation at each abutment.

For the steel end of bridge detail, refer to Figure 6.2-2. Consider the following, for end of girder details:

- Place stiffener as close to the centerline of bearing as practical.
- Provide holes in girder for reinforcing steel according to design.

The following construction considerations should be specified in the plans:

- The anchor bolts shall be checked for plumb;
- Ensure the anchor bolts have been grouted properly;
- Prior to placing the girders, the elevation for the leveling plates shall be check;
- For each girder, the contractor shall grease the top surface of the leveling plate at the highest elevation; and
- Once girders have been placed, check the anchor bolts for plumb and wipe all excess grease from the leveling plate (if for any reason, the anchoring bolts are out of plumb; this will need to be reported to the Engineer for stability determination)

The construction joint shall be sloped to match the crown of the roadway surface of the bridge deck. This will allow for a single bearing device to be used at all bearing locations. The plans should show elevations at each end and at the centerline peak of the pile cap. Detailing the construction joint as level will require different heights for the anchor bolts and may cause the bolt to buckle.

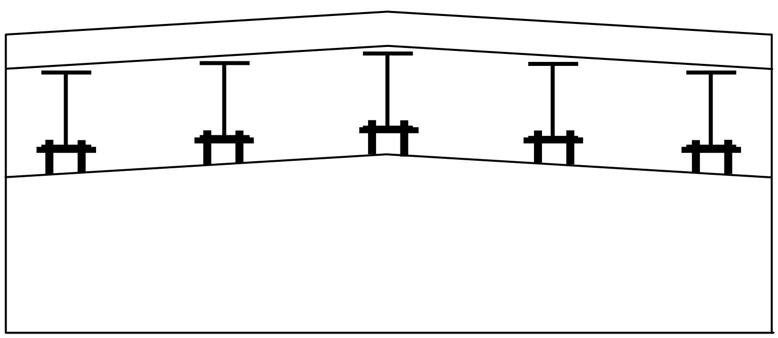

Figure 6.2-1 Cross Slope of Pile cap must match the cross slope of the deck.

1.1.1.1

moisture; galvanizing the leveling plate will increase the friction between that and the girder, the Structures Section is trying to avoid.

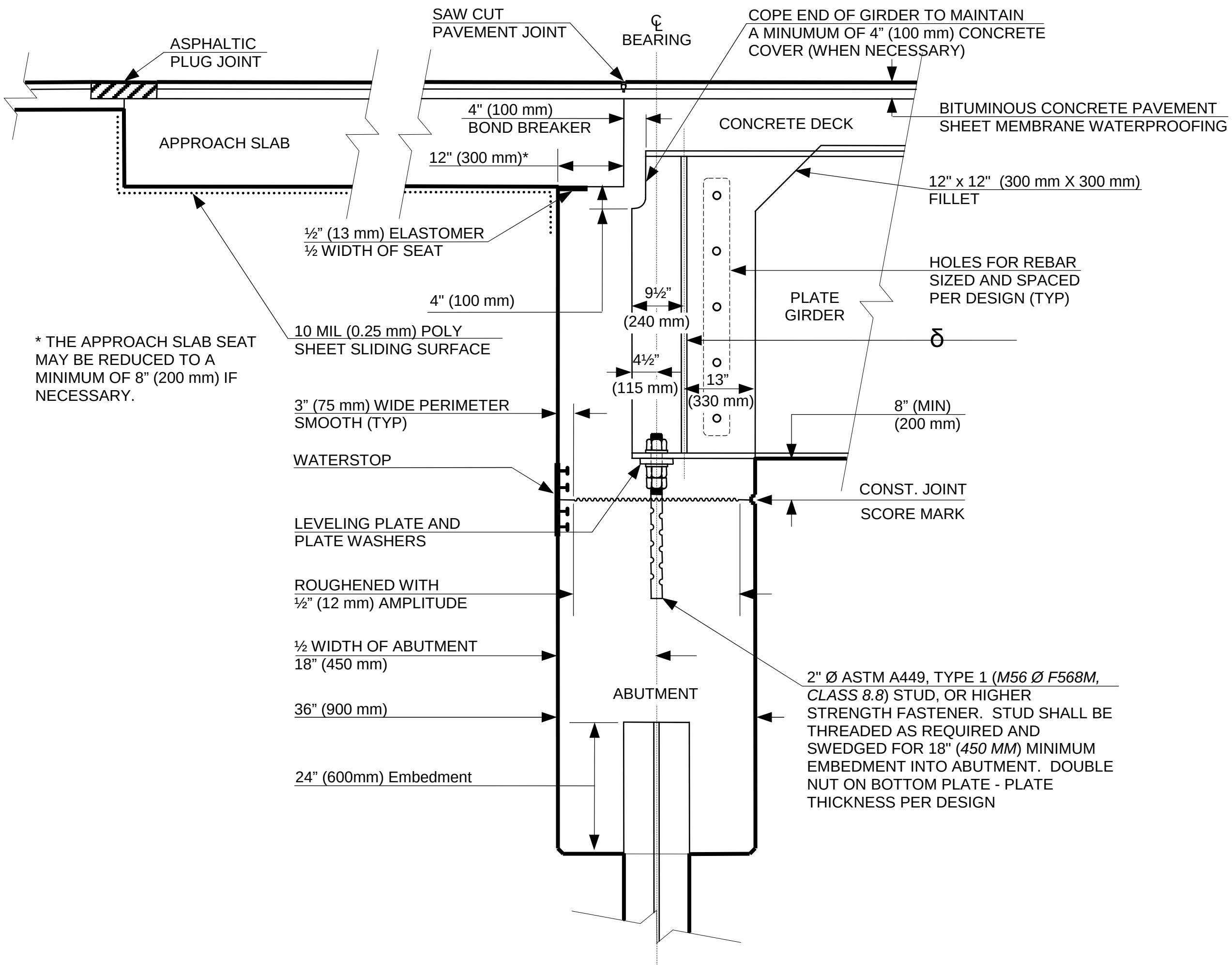

Figure 6.2-2 Bridge End Detail Type **A** (Steel Girder)[2]

[2] **Changed July 15, 2009 – Bridge End Detail "J" has been changed to "A" in the 2009 LRFD Structures Manual.**

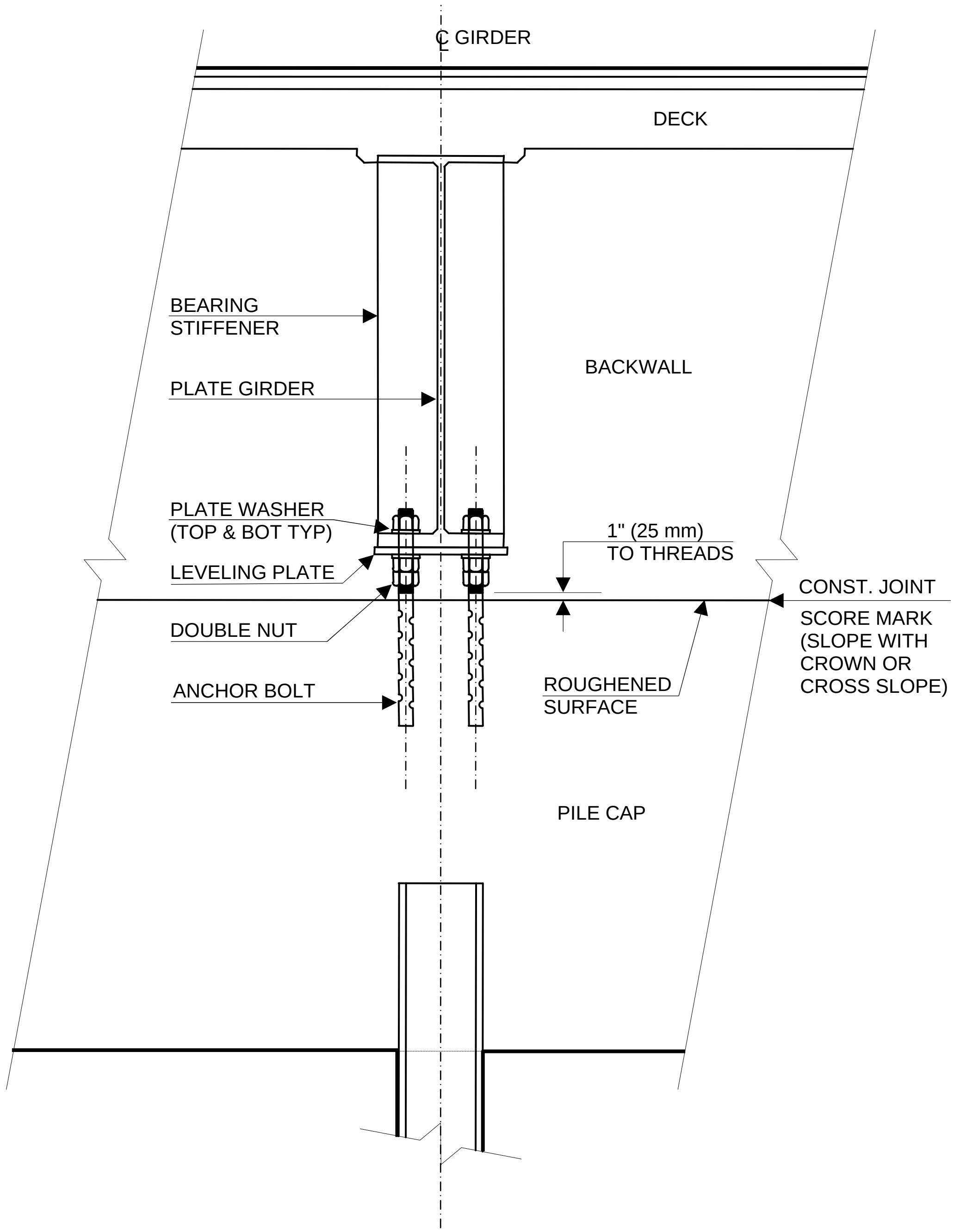

Figure 6.2-3 Bearing Assembly Cross Section (Steel Girder)

SECTION 7 ALUMINUM STRUCTURES

7.1 GENERAL INFORMATION

Refer to the Structures Manual for more information.

SECTION 8 WOOD STRUCTURES

8.1 GENERAL INFORMATION

Refer to the Structures Manual for more information.

SECTION 9 DECK AND DECK SYSTEMS

9.1 GENERAL INFORMATION

Refer to the Structures Manual for more information.

SECTION 10 FOUNDATIONS

10.1 INITIAL CONSIDERATIONS

Use this section with LRFD Section 10. The contents of this section focus on the material in the LRFD Specifications that relates to the *Simplified Design Method* for integral abutment design. Generally, this section relates to piles driven through non-cohesive soils to bedrock. Design issues including skin friction, settlement, and downdrag have not been fully explored in this guideline. If these issues are a concern, review LRFD Section 10 and more guidance consult with the Soils & Foundations Engineer at Materials and Research.

10.1.1 Geotechnical Exploration

If bedrock is encountered, it is important to obtain, with some degree of accuracy, the bedrock profile along the row of piles at each abutment location. If practical, borings should be requested prior to Conceptual Plans, while in project scoping, to determine the feasibility of using integral abutments. In lieu of borings, other sources of subterrain information are available to determine existing foundation conditions include reviewing record plans for the existing bridge and conducting a site investigation. Well logs from the Agency of Natural Resources (ANR), which provide depth to bedrock data, are another good source for this information.

10.1.2 Pile Design and Verification

The designer will design the pile for structural resistance to applied loads and displacements. For the most part, this resistance will be based on strength limit state requirements (Section 10.4). With the resistance of the pile determined, the designer will need to determine what method will be used to monitor the driven pile (See Section 10.5). Monitoring the piling driving process will help prevent damage to the pile, thereby ensuring the pile's resistance to a reasonable level of certainty.

10.1.3 Required Information for Contract Documents

Refer to LRFD Section 10.7.1.5 for specific design requirements of piles used in integral abutments. In addition to the requirements of the LRFD specifications, the plans should include the:

- Nominal Axial Pile Resistance (NAPR) (see Section 4.5.2),
- the type and size of pile required to provide adequate support,
- number and location of each pile,
- minimum pile tip elevation necessary to satisfy the requirements caused by uplift, scour, downdrag, settlement, liquefaction, lateral loads, and seismic conditions,
- pile quantity estimation from estimated pile penetration required to meet the NAPR and other design requirements, and
- number and type of pile load tests required.

10.2 SELECTING A PILE FOR INTEGRAL ABUTMENTS

This guideline only addresses those issues that one would typically see in an Integral Abutment structure. Any variation from the underlying assumptions used in this document must be addressed by the Engineer.

10.2.1 Loads on Piles

Refer to LRFD Section 10.7.1.6 for loading requirements. This guideline addresses all vertical dead and live loads (P_u), lateral displacements (δ) and moments (M_u) caused by superimposed dead loads and live loads applied to the pile. Downdrag and uplift, which are not addressed in this guideline, should be considered independently for each project as it is required. Special service load analyses shall be considered for each site as required by the expected site conditions. Refer to LRFD Section 10.7.2 for more information. The loads (Q_i) are factored as specified in LRFD Section 3.4.1.

$$P_u = \sum \gamma_i Q_i \qquad (10.2.1\text{-}1)$$

10.2.2 Pile Cap Geometry

Refer to LRFD Section 10.7.1.2 for minimum clearances for placing piles. The overall shape of the pile cap in plan should be trapezoidal[1], with the front and back faces parallel and 3 ft (*900 mm*) apart and extending to the width of the roadway plus the shoulders and wingwall thicknesses taking into consideration skew and the wingwall configuration (See Section 2.2.1). The requirements in this section will be met when using the details in Section 4 of this guideline.

The required 6 inch (*150 mm*) minimum cover to the front or back face of the pile cap should be a concern only when piles are driven out of specified tolerance during construction.

10.2.2.1 Number of Piles and Pile Spacing

Generally for beam–deck systems, the number of piles at each abutment will equal the number of beams. The spacing of the piles for these bridges will generally be the same as the beam spacing. The designer may use fewer higher resistance piles to save on construction costs.

Bridge types such as cast-in-place concrete or butted prestressed deck beams will require a pile spacing that will ensure adequate resistance while using the fewest number of piles.

For the H-piles to be designed as single piles the minimum pile spacing should be equal or greater than 5 ft – 10 inch. Piles spaced closer then this may need to be designed as a group (See Section 10.2.2.2). Each abutment shall have a minimum of four piles.

10.2.2.2 Pile Groups

The LRFD Specifications offers very little guidance on what constitutes a pile group. LRFD Section 10.7.2.4 has some guidance to pile groups as it relates to movement. The section requires a group reduction factors to be applied once pile spacing becomes closer then 5 times the pile diameter (or width). For H-piles with flange widths of 12 and 14 inches piles may be considered a pile group when the pile spacing is less than 5 ft and 5 ft – 10 inches (*1525 mm and 1780 mm*), respectively. Therefore, if the pile spacing exceeds these values then each pile can be designed as a single pile.

10.2.2.3 Pile Length Requirement

The pile length will typically be what is required for end bearing on bedrock. Refer to LRFD Section 10.7.3.3 when other support criteria will be used. The *Simplified Design Method* for integral abutments is appropriate when the depth to bedrock or refusal layer is 16 ft (*5 m*) or deeper or where projected scour depth reduces the embedment length to at least 16 ft (*5 m*). Depths less than this will require a more detailed analysis.

10.3 SERVICE LIMIT STATE

Refer to LRFD Section 10.5.2 for Service Limit State requirements. If piles will be driven to bedrock, settlement will not be a concern. All bridge designs need to consider horizontal movement, overall stability and scour at the design flood elevation.

10.4 STRENGTH LIMIT STATE

The following issues shall be considered during design. Refer to LRFD Section 10.7.3.1 for more information.

- The Nominal Structural Pile Resistance (NSPR) of the pile and/or pile group (see Section 4.5.2).

[1] A trapezoid is a quadrilateral, which is defined as a shape with four sides, which has at least one set of parallel sides. This infers that rectangles and parallelograms are subsets of the trapezoid shape. (*Wikipedia, 2008*)

- If a minimum pile tip elevation is required for the particular site condition and load, it shall be determine based on the maximum (deepest) depth needed to meet all of the applicable requirements identified in LRFD Section 10.7.6.

10.4.1 Nominal Structural Pile Resistance (NSPR)

The Nominal Structural Pile Resistance (NSPR) (P_n) is the limiting structural axial compressive resistance of a pile for the structural limit state, and according to LRFD Section 10.7.3.13, shall be calculated using LRFD Section 6.9.4.1 for noncomposite piles with the resistance factors specified in LRFD Section 6.5.4.2 and Section 6.15 for severe driving conditions (see Section 4.5.2). This limit is dependant on the unbraced length. The lower zone of the pile is considered to be fully embedded; therefore λ shall be taken as 0.

The effective length of laterally unsupported piles may be estimated using the provisions in LRFD Section 10.7.3.13.4; however, using L-Pile® with the anticipated lateral displacement and axial load from the superstructure may provide a more realistic value for the effective length of the pile.

10.4.2 Nominal Axial Pile Resistance (NAPR)

LRFD Section 10.7.3.2.3 requires that the Nominal Axial Pile Resistance (NAPR) R_n of piles driven to bedrock, where pile penetration into the bedrock formation is minimal, be equal to the factored axial load divided by the resistance factor for compression (ϕ_c) and limited by the NSPR (see Section 10.4.1).

$$R_n = \frac{P_u}{\phi_c} \le P_n \tag{10.4.2-1}$$

10.4.3 Downdrag and Other Losses to Geotechnical Strength

The designer will need to review the Geotechnical Report for downdrag and liquefaction considerations. Further review of the Hydraulics Report will provide the scour parameters. These parameters reduce the available skin frictional resistance of the pile. Downdrag (DD), in particular will also affect the axial resistance of the pile. If downdrag is a concern, the NAPR (R_n) will need to be adjusted to account for this load effect.

$$R_n = \frac{P_u}{\phi_c} + DD \le P_u \tag{10.4.4-1}$$

10.4.4 Strength Limit State Resistance Factors for Driven Piles

Refer to LRFD Section 10.5.5.2.3 for more information on this section.

When driving piles to bedrock, use the resistance factors for the Strength Limit State defined in LRFD Sections 6.5.4.2 and 6.15.2 for severe pile driving conditions.

Generally the following list of resistance factors may be used if the project meets the corresponding assumptions:

- For Strength Limit State (NSPR) use the resistance factors from LRFD 6.5.4.2:
  - Use ϕ_c = 0.50 for axial resistance of H-piles in compression and subject to damage due to severe pile driving conditions where use of a pile tip is necessary. This is usually the lower zone of piles driven to bedrock.
  - For combined axial and flexural resistance of the upper zone of piles:
    - axial resistance: ϕ_c = 0.70
    - flexural resistance: ϕ_f = 1.00
  - Use ϕ = 1.00 for resistance of the entire pile during pile driving.

10.5 PILE DRIVING ANALYSIS

This section provides guidance in selecting an appropriate pile driving criteria to ensure the driven pile will have the desired design resistance. In addition to what is required for the plans (see Section 10.1.3), the maximum pile driving stress will need to be determined to prevent overdriving the pile (See Section 10.5.2). Refer to LRFD Section 10.7.3.1 for more information.

The following will be determined during construction by the Geotechnical Engineer.

- The drivability of the selected pile to achieve the required Nominal Axial Pile Resistance (NAPR) or minimum penetration with acceptable driving stresses at a satisfactory blow count per unit length of penetration. This check is done during construction by the Geotechnical Engineer once pile driving information is received from the Contractor.
- The Nominal Pile Driving Resistance (NPDR) expected in order to reach the minimum pile penetration required, if applicable, including any soil/pile skin friction that will not contribute to the long term nominal axial resistance of the pile, e.g., soil contributing to downdrag, or soil that will be scoured away. (see Section 10.5.3).

10.5.1 Pile Driving Concerns

When driving piles to bedrock, consider the following (see LRFD Commentary Section C10.7.3.2.3):

- Use care in driving piles to hard bedrock to avoid pile tip damage.
- Protect the tips of steel piles driven to hard bedrock by high strength, cast steel tips with teeth, see the Vermont Standard Specifications 505.04(e).

Select the ϕ_{mon} factor from Table 10.5-1 for the monitoring method used.

Where pile friction resistance is being counted on, other conditions will need to be addressed as specified in the LRFD Specifications.

Table 10.5-1 Common ϕ_{mon} Factors for Pile Driving[2]

Test	ϕ_{mon}
Dynamic Test (ϕ_{dyn})	0.65
Wave equation analysis, w/o monitoring (ϕ_{stat})	0.40
Static Analysis - End Bearing in Bedrock (ϕ_{stat})	0.45

10.5.2 Maximum Pile Driving Stress

The plans should establish a pile driving criteria. The Engineer will be required to perform a drivability analysis using a wave equation analysis. The stresses anywhere in the pile during driving shall not exceed the maximum driving stress (σ_{dr}). For steel H-Piles, ϕ_{da} equals ϕ for resistance during pile driving (see LRFD Sections 10.7.8, 10.5.5.2.3 and 6.5.4.2). This analysis is performed by the Geotechnical Engineer for each pile driving project. First, during design as a feasibility check and then later when the Pile Driving Equipment Data Form (PDEDF) has been submitted by the Contractor.

$$\sigma_{dr} = 0.9\phi_{da}F_y \qquad (10.5.2\text{-}1)$$

The resistance factor, ϕ_{da} for driving steel H-piles equals 1.00; therefore σ_{dr} will equal 90% of F_y. With the requirement of grade 50 (*345*) steel for steel piles, σ_{dr} will equal 45 ksi (*310 MPa*) in most cases.

[2] See LRFD Section 10.5.5.2 for other selections.

10.5.3 Nominal Pile Driving Resistance (NPDR)

The Nominal Pile Driving Resistance (NPDR) R_{ndr}, also referred to as the Geotechnical Resistance, is the required nominal resistance the pile will be driven to. This capacity is based on the applied load adjusted for the method used to verify the NAPR. This value is the result of dividing the applied axial load by the pile monitoring resistance factor ϕ_{mon} (See Section 10.5.3.1). When driving a pile to bedrock, the NPDR should not exceed the NSPR.

$$R_{ndr} = \left(\frac{P_u}{\phi_{mon}} \right) \tag{10.5.3-1}$$

$$R_{ndr} \le P_n \tag{10.5.3-2}$$

10.5.3.1 Verification of the Nominal Axial Pile Resistance (NAPR) in Compression

Refer to LRFD Section 10.7.3.8.1 for more information on this section.

The NAPR (R_n) should be field verified during pile installation. The verification resistance factor (ϕ_{mon}) shall be based on the monitoring method used to verify pile axial resistance as specified in LRFD Section 10.5.5.2.3 (see Table 10.5-1 and Section 10.5.4).

Pile tips are often used to protect the piles from driving damage. To further prevent pile damage, a pile-driving acceptance criterion shall be developed. Though static analysis may be used to verify the resistance of most driven piles, dynamic measurements should be used to monitor for pile damage when the NAPR exceed 600 kips (*2500 kN*) (As required by LRFD Section 10.7.3.2.3).

A static load test should only be considered for an integral abutment bridge when extreme conditions dictate that such a test should be performed. In almost every case, it would be far more economical to use larger piles rather then pay for a static load test. Extreme conditions requiring static tests may include: installing a large structure that will put heavy loads on the pile; driving piles through dense to very dense soil; or when there is a limit on the number of piles that can be driven for each substructure. In any of these cases, piles will need to be driven close to their resistance. Most monitoring will be done by wave equation, static analysis, dynamic testing or dynamic formula – in that order.

The Geotechnical Engineer, in recent years, has been requiring the use of two dynamic pile tests on each project as a means to collect data on pile driving in Vermont. The engineer should be cautioned that two dynamic pile tests do not satisfy LRFD Section 10.5.5.2.3. LRFD requires an absolute minimum of three dynamic pile tests when driving 25 piles or less in a single site with low geotechnical variability. In situations when each abutment can be considered a single site as defined by LRFD, or if there is significant geotechnical variability at the site, the required number of test for a site could go as high as 6 tests. Considering that most integral abutment bridges would have 4, 5 or 6 piles per abutment, one could conclude that the requirements of LRFD Section 10.5.5.2.3 are not compatible with those of integral abutments. Currently, the Agency is developing a policy[3] regarding using the resistance factor (ϕ_{dyn}) of 0.65 for two dynamic pile tests for each project.

10.5.4 Resistance Factors for Verifying the NAPR

The factor (ϕ_{mon}) to use for verifying the NAPR shall be selected from Table 10.5-1. Generally the following list of resistance factors may be used if the project meets the corresponding assumptions:

- For NPDR analysis, use the resistance factors (ϕ_{da}) from Section 10.4.4.
- NAPR is being verified by Static Load Tests for each abutment. (Not an economical choice)
  - Low site variability – ϕ_{dyn} = 0.90
  - Medium site variability – ϕ_{dyn} = 0.75
- NAPR is being verified by 3 to 6 dynamic pile tests, depending on site conditions, when using up to 7 piles

[3] At the printing of this guideline the policy on the resistance factor ϕ_{dyn} has yet to be completed.

per abutment or 5 piles per abutment and a pier – ϕ_{dyn} = 0.65.

- NAPR is being verified by static analysis with bearing on bedrock – ϕ_{stat} = 0.45

10.6 DESIGN STEPS FOR PILES

The following steps shall be followed for the design and verification of the pile design for integral abutment bridges. Refer to Appendix B for a design example that follows this outline:

- Determine foundation displacements (δ) and load effects (P_u and M_u) from the superstructure and substructure designs (LRFD Section 10.7.1.6).
- If applicable, determine the magnitude of scour and liquefaction susceptibility (Refer to the Hydraulics and Geotechnical Reports).
- Determine if downdrag loadings (DD) should be considered. (Refer to the Geotechnical Report).
- Select preliminary pile size and pile layout (LRFD Section 10.7.1.2).
- Estimate Pile Length (typically to bedrock).
- Determine the Nominal Structural Pile Resistance (NSPR) (P_n) (LRFD Section 6.9.4.1).
- Determine Structural Flexural Resistance (H-Pile Weak Axis – LRFD Section 6.12.2.2).
- Run L-Pile to determine pile design unbraced lengths (l_b) and internal moments.
- Determine resistance factors (ϕ_c and ϕ_f) for the structural strength for the upper and lower zone of the pile (LRFD Sections 6.15.2 and 6.5.4.2).
- Determine the Nominal Axial Pile Resistance (NAPR) (R_n) (Applied loads, P_u, divided by ϕ_c).
- Determine if the applied moment on the pile will cause the pile head to enter plastic deformation by using the interaction of the combined axial & flexural load effects of a single pile (LRFD Section 6.9.2.2).
- Maximum shear load effect for a single pile (Refer L-Pile® Results).
- Determine Structural Shear Resistance (H-Pile Weak Axis – AISC G7).
- Determine method for pile driving acceptance criteria (LRFD Section 10.5.5).
- Determine Resistance Factor for Geotechnical Strength (ϕ_{mon}) (LRFD Section 10.5.5.2.3).
- Determine Nominal Pile Driving Resistance (NPDR) or also called the Geotechnical Resistance for a single pile (LRFD Section 10.7.3.13).
- Review the final design with L-Pile®.
- Show proper Pile Data on Plan Sheets.

SECTION 11 ABUTMENT, PIERS AND WALLS

11.1 GENERAL INFORMATION

Refer to the Structures Manual for more information.

SECTION 12 BURIED STRUCTURES AND TUNNEL LINERS

12.1 GENERAL INFORMATION

Refer to the Structures Manual for more information.

SECTION 13 RAILINGS

13.1 GENERAL INFORMATION

Refer to the Structures Manual for more information.

SECTION 14 JOINTS AND BEARINGS

14.1 GENERAL INFORMATION

Refer to the Structures Manual for more information.

SECTION 15 SUMMARY

Jointless bridges have proven to be significant sources of cost savings in bridge programs across the nation. The implementation of a bridge program that supports and encourages the use of jointless bridges as the primary design option will inevitably save thousands of dollars and allow for better uses of the program's economic resources. Savings experienced in both initial construction and reduced maintenance costs will translate into additional bridges being built, resulting in an improved integrated transportation system responsible for the transport of goods and services in a safe, efficient, cost effective, and environmentally sensitive manner.

A primary Committee objective has been to keep the Agency's vision and mission statements in the forefront when developing effective design guidelines for future projects. However, it is recognized that the success of future projects is highly dependent upon a careful analysis and monitoring of these types of structures. Therefore, the Integral Abutment Committee has supported the RAC funded Performance Monitoring of Jointless Bridges research project. This research project consists of three phases. Phase I and II of this project have been completed. Phase I was a literature search, while Phase II consisted of the development of an instrumentation plan based on Phase I conclusions and recommendations. Phase III consists of the actual monitoring of the instrumentation (installed by the Consultant) as approved by the Committee. The Committee hopes to provide instrumentation for three integral abutment bridges (two in-line straight girder and one curved girder) as part of this research project.

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Section 4 ? Substructures, 4.8 Jointless Bridges with Curtain Wall Abutments

APPENDIX A DESIGN OUTLINE

Basic Steps for integral abutment bridge design

Step 1 – Conceptual Design

Determine:

- Bridge Location and Geometry
- Initial Attempt at Span and Typical

Obtain:

- Initial Hydraulic Information
- Scour Depth

Step 2 – Preliminary Design

Refine Geometry

- Backwall Depth
- Pile Cap Depth
- Required Elevations

Finalize Deck Layout

- Typical Section
- Rail
- Girder Section
- Approach Slab

Perform a preliminary deck design assuming a simply supported deck

Step 3 - Loads

Determine

- Dead Load (DL) and any P-Delta Loads on Piles before placement of backwall.
- Dead Load (DL, DW) and Live Load (LL) after Deck cures.
- Thermal, Creep and Shrinkage effects.
- Vertical and Longitudinal Load Effects (Breaking)
- Tabulate Appropriate Load Combinations and Load Factors.

Step 4 – Preliminary Pile Design[1]

Determine

- Pile Orientation (typically weak axis).
- Size Pile (use recommended shapes in guidelines)

[1] See Step 4 in the Design Example in Appendix for more detail.

- Determine the Number of Required Piles. (Typically 1 pile at abutment for each girder. Use guidelines to approximate.)
- Prebore Requirements
- Estimate Target Construction Period

Step 5 – Geotechnical Report

Obtain the Geotechnical Report

Step 6 – Pile Design Parameters[2]

L-Pile Analysis

- Construction Condition
- Finished Condition
- Scour Condition

Need for each condition:

- Depth to Fixity
- Pile Top Moment (Max=Mp)
- Unbraced Length

Step 7 – Check Pile Design[3]

Determine:

- Live Load Moment and Rotation
- Interaction Check for the Second Segment
- Check Pile Capacity

Step 8 – Construction Checks

- Check Temporary Bearings
- Check Bearing Stiffeners for Vertical Loads
- Consider other Construction Conditions.

Step 9 – Backwall/Pile Cap Design

- Design Backwall/Pile Cap (Strut and Tie)
- Passive Earth Pressure
- Bending/shear between Girders (Horizontal)
- Pullout/Punch-through
- Bending/Shear at Scour (Vertical)
- Pile Embedment Design (Standard Detail)

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[2] See Step 6 in the Design Example in Appendix for more detail.
[3] See Step 7 in the Design Example in Appendix for more detail.

Step 10 – Finalize Deck Design

Design Negative Moment Region at End of Deck.

Step 11 – Finalize Substructure Design

Design Wingwalls

APPENDIX B DESIGN EXAMPLE

This design example requires a completed superstructure design. The vertical load of the superstructure will need to be distributed over the piles. This example will assume that the vertical load is evenly distributed for simplicity. The superstructure longitudinal displacements will also need to be determined. Longitudinal displacements typically include effects caused by temperature increases and decreases, creep, shrinkage or displacements caused by longitudinal forces. Additional analysis may be done for lateral displacements caused by lateral load effects. This example will look at a single direction longitudinal displacement for simplicity.

This example will ignore the fixed end moments caused by live load and composite dead loads as these may have a marginal negative effect for deck elongation and a beneficial effect for deck contraction. For a more precise analysis, these load effects will need to be combined with the moment caused by displacement. To calculate the end moments caused by the above mentioned effects, a pile is required; therefore, it can be concluded that this process will need to be iterative.

This example uses an L-Pile analysis to determine the unbraced lengths of the different pile segments, maximum moments both at the top of the pile and in the second segment, depth to fixity and shear at the top of the pile. A MathCAD® sheet has been developed for the design calculations. This example will be using this sheet to demonstrate the calculations required for the pile design.

This design example picks up at step four in Appendix A. All previous steps should have been completed. All the yellow highlighted values are collected from either previously calculated steps or results from design or analysis software.

Step 4 – Preliminary Pile Design

- Pile orientation will be weak axis bending
- Determine Pile Size
    - Required:
        - Factored applied superstructure and substructure vertical dead and live load (P_u) distributed to each pile.

$$P_u = 416795.74\ lbs = 416.796 \cdot kips \tag{4-1}$$

        - Steel pile strength: F_y = 50 ksi; E = 29000 ksi.
    - Select resistance factors (ϕ) [4.5.2]
        - ϕ_{cl} = 0.50 for the lower zone of the pile.
        - ϕ_{cu} = 0.70 for the upper zone of the pile.
        - ϕ_f = 1.00 for the lower zone of the pile.
    - Determine the required Nominal Axial Pile Resistance (NAPR) [**10.4.2**]

$$R_{n.upper} = \frac{P_u}{\phi_{cu}} = 595.422 \cdot kips \tag{4-2}$$

$$R_{n.lower} = \frac{P_u}{\phi_{cl}} = 833.591 \cdot kips \tag{4-3}$$

$$R_n = max\left(R_{n.upper}, R_{n.lower}\right) = 833.591 \cdot kips \tag{4-4}$$

    - With the required NAPR, estimate the required pile area. [**4.5.1.5**]

$$A_{s.req} = \frac{R_n}{0.80 \cdot F_y} = 20.84\ in^2 \tag{4-5}$$

- Select a H-Pile with an area of $A_{s.reg}$ or higher. Choose a HP 12X74 (A_s = 21.8 inch^2).

- Check the Pile Properties [LRFD 6.9.4.2]

$$b_f = 12.2 \cdot inch \tag{4-6}$$

$$t_f = 0.61 \cdot inch \tag{4-7}$$

$$d = 12.1 \cdot inch \tag{4-8}$$

$$k = 1.313 \cdot inch \tag{4-9}$$

$$h = d - 2 \cdot k = 9.475 \cdot inch \tag{4-10}$$

$$\lambda_f = \frac{b_f}{2 \cdot t_f} = 10 \tag{4-11}$$

$$\lambda_w = \frac{h}{t_w} = 15.661 \tag{4-12}$$

$$k_f = 0.56 \tag{4-13}$$

$$k_w = 1.49 \tag{4-14}$$

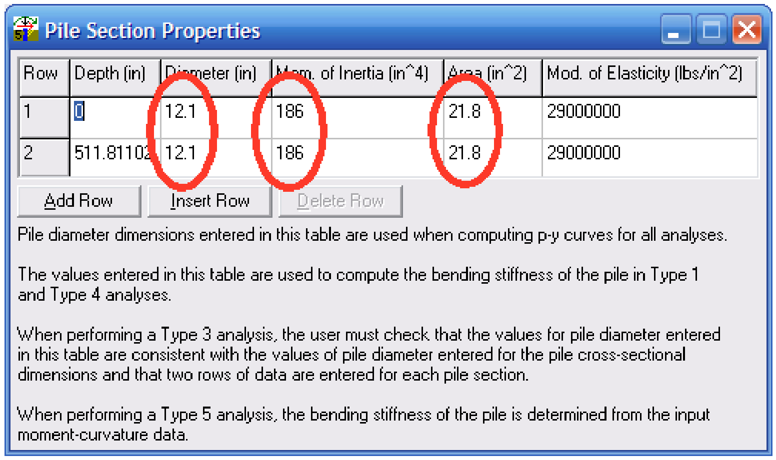
$$Check\left[\left(\lambda_f \le k_f \cdot \sqrt{\frac{E}{F_y}}\right) \wedge \left(\lambda_w \le k_w \cdot \sqrt{\frac{E}{F_y}}\right)\right] = \text{"OK"} \tag{4-16}$$

<u>Step 5 – Geotechnical Report</u>

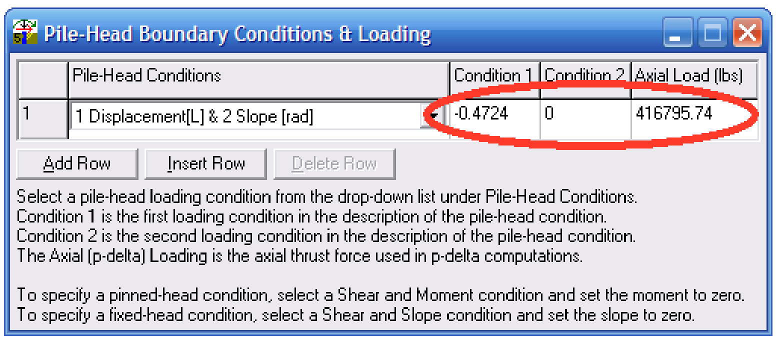
- At this point, order a Geotechnical Analysis of the site and request an L-Pile® data file to be created. A Final Geotechnical Report will be sent upon completion.

<u>Step 6 – Pile Design Parameters</u>

- Enter the selected pile into the L-Pile® data file using the following data:
  - Area of pile: A_s = 21.8 in^2
  - Diameter: d = 12.1
  - Moment of inertia: I_y = 186 inch^4



- Enter the following load case data:
  - Condition 1 – Displacement δ = -0.4724 inches (*from design*)
  - Condition 2 – Rotation θ = 0° (*Assuming no rotation*)



  - Axial Load P_u = 416795.74 lbs (*from design*)
- Run the L-Pile® analysis.
  - Obtain the maximum moment at the top of the pile from the output.

SEGMENT OF L-PILE® OUTPUT:

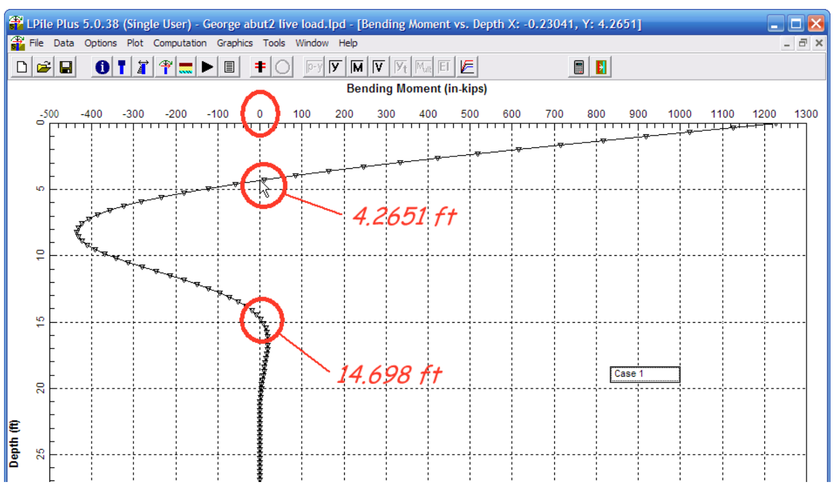
```
. . .
------------------------------------------------------------------------------
             Computed Values of Load Distribution and Deflection
                 for Lateral Loading for Load Case Number  1
------------------------------------------------------------------------------

Pile-head boundary conditions are Displacement and Slope (BC Type 5)
Specified deflection at pile head   =        -.472400 in
Specified slope at pile head        =       0.000E+00 in/in
Specified axial load at pile head   =      416795.740 lbs

  Depth    Deflect.     Moment      Shear       Slope      Total       Soil Res.      Es*h
    X         y           M           V           S        Stress         p           F/L
   in        in        lbs-in        lbs         Rad.     lbs/in**2    lbs/in       lbs/in
-------- --------- ----------- ----------- ----------- ----------- ----------- -----------
   0.000  -.472400   1227727.  -25865.7550      0.0000  59053.2155      0.0000      0.0000
   3.937  -.470636   1125401.  -25742.7766    .0008588  55724.8399     31.2227    261.1868
   7.874  -.465638   1022210.  -25546.9805    .0016425  52368.3634     68.2694    577.2231
  11.811  -.457703    918853.  -25201.1718    .0023509  49006.4836    107.4015    923.8319
  15.748  -.447127    816060.  -24704.1438    .0029840  45662.9564    145.0887   1277.5233
. . .
```

$$M_{u.top} = 1227.727\ inch \cdot kips \qquad (6\text{-}3)$$

- o Obtain the unbraced lengths of the top segment and the second segment of the upper zone of the pile. This can be done by interpolating the depth when the moment values cross from positive to negative, or use the graphical out put from L-Pile® to estimate the depth. By moving the cursor over the graph, the corresponding Bending moment vs. depth displays at the title bar of L-Pile®.



$$l_{b.top} = 4.2651\ ft = 51.181 \cdot inch \qquad (6\text{-}1)$$

$$l_{b.2nd} = 10.4329\ ft = 125.195 \cdot inch \qquad (6\text{-}2)$$

- o Calculate the normalized column slenderness factor (λ) for each segment [LRFD 6.9.4.1]
  - ▪ For K values refer to LRFD Commentary Table C4.6.2.5-1.
- o For the top segment – fixed at top, pinned at bottom:

$$K_{top} = 1.2 \qquad (6\text{-}3)$$

$$\lambda_{top} = \left(\frac{K_{top} \cdot l_{b.top}}{r_y \cdot \pi}\right)^2 \cdot \frac{F_y}{E} = 0.077 \tag{6-4}$$

- For the second segment pinned at top and bottom:

$$K_{2nd} = 1 \tag{6-5}$$

$$\lambda_{2nd} = \left(\frac{K_{2nd} \cdot l_{b.2nd}}{r_y \cdot \pi}\right)^2 \cdot \frac{F_y}{E} = 0.321 \tag{6-6}$$

- Calculate the Nominal Structural Pile Resistance (NSPR) for both segments and the lower zone of the pile:

$$P_{n.top} = \begin{vmatrix} \left(0.66^{\lambda_{top}} \cdot F_y \cdot A_s\right) & \text{if } \lambda_{top} \le 2.25 \\ \dfrac{0.88 \cdot F_y \cdot A_s}{\lambda_{top}} & \text{otherwise} \end{vmatrix} = 1055.576 \cdot kips \tag{6-7}$$

- Interpolating from Table 4.5.1.5-4 (K=1.2) with l_b = 4.2651 ft $P_{n.top}$ = 1055.10 kips.

$$P_{n.2nd} = \begin{vmatrix} \left(0.66^{\lambda_{2nd}} \cdot F_y \cdot A_s\right) & \text{if } \lambda_{top} \le 2.25 \\ \dfrac{0.88 \cdot F_y \cdot A_s}{\lambda_{2nd}} & \text{otherwise} \end{vmatrix} = 953.928 \cdot kips \tag{6-8}$$

- Interpolating from Table 4.5.1.5-3 (K=1.0) with l_b = 10.4329 ft $P_{n.2nd}$ = 952.96 kips.

$$P_{n.bottom} = 0.66^0 F_y \cdot A_s = 1090 \cdot kips \tag{6-9}$$

(λ = 0 for fully braced pile)

- This value can be obtained from Table 4.5.1.5-1.
- Calculate the Structural Pile Resistance (P_r)

$$P_{r.top} = \phi_{cu} \cdot P_{n.top} = 738.903 \cdot kips \tag{6-10}$$

$$P_{r.2nd} = \phi_{cu} \cdot P_{n.2nd} = 667.75 \cdot kips \tag{6-11}$$

$$P_{r.bottom} = \phi_{cl} \cdot P_{n.bottom} = 545 \cdot kips \tag{6-12}$$

- Check to make sure the pile size is not too big. The ratio of the applied load to the structural resistance should not be less then 0.2.

$$\frac{P_u}{P_{r.top}} = 0.564 \tag{6-13}$$

$$\frac{P_u}{P_{r.2nd}} = 0.624 \tag{6-14}$$

$$Check\left[\left(\frac{P_u}{P_{r.top}} > 0.2\right) \wedge \left(\frac{P_u}{P_{r.2nd}} > 0.2\right)\right] = \text{"OK"} \tag{6-15}$$

- Since the lower zone of the pile will have virtually no moment, the entire section can carry the required vertical loads. Make sure the applied load will not exceed the resistance of the lower zone.

$$\frac{P_u}{P_{r.bottom}} = 0.765 \tag{6-16}$$

$$Check\left(\frac{P_u}{P_{r.bottom}} < 1\right) = \text{"OK"} \tag{6-17}$$

- At this point calculate the Nominal Flexural Resistance

$$\lambda_{pf} = 0.38 \cdot \sqrt{\frac{E}{F_y}} = 9.152 \tag{6-18}$$

$$\lambda_{rf} = 0.83 \cdot \sqrt{\frac{E}{F_y}} = 19.989 \tag{6-19}$$

$$M_n = \begin{cases} \left(F_y \cdot Z_y\right) & \text{if } \lambda_f < \lambda_{pf} \\ \left[\left[1 - \left(1 - \frac{S_y}{Z_y}\right) \cdot \left(\frac{\lambda_f - \lambda_{pf}}{0.45 \cdot \sqrt{\frac{E}{F_y}}}\right)\right] \cdot F_y \cdot Z_y\right] & \text{if } \lambda_{pf} < \lambda_f < \lambda_{rf} \\ \text{"Select another Pile"} & \text{otherwise} \end{cases} \tag{6-20}$$

$$M_n = 2266.95 \cdot inch \cdot kips \tag{6-21}$$

- Then calculate the Flexural Resistance

$$\phi_f = 1 \tag{6-22}$$

$$M_r = \phi_f \cdot M_n = 2266.95 \cdot inch \cdot kips \tag{6-23}$$

- Finally Check the Moment that will cause a plastic hinge at the pile cap.

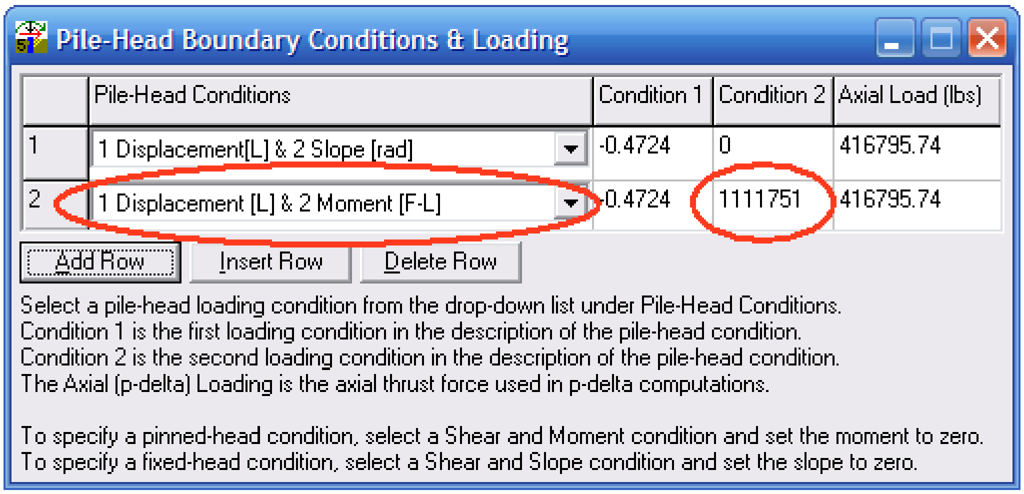
$$M_{p'} = \frac{9.0}{8.0}\left(1 - \frac{P_u}{P_{r.top}}\right) \cdot M_r = 1111.751 \cdot inch \cdot kips \tag{6-24}$$

$$M_{p'} = 92.646 \cdot ft \cdot kips \tag{6-25}$$

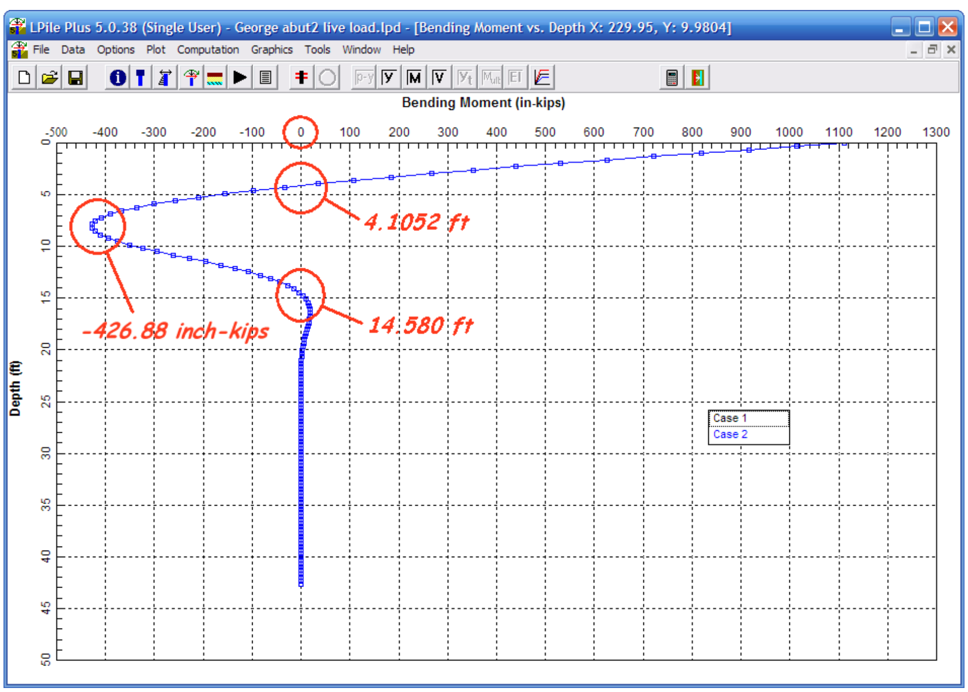
$$M_{u.top} = 1227.727\ inch \cdot kips$$ (See 6-3)

- o Since the Applied moment exceeded the moment that would create a plastic hinge, it can be assumed that the pile head has entered plastic deformation and therefore the moment that can be applied to the pilehead cannot exceed $M_{p'}$.

Step 7 – Check Pile Design



- At this point, create a new load case in L-Pile® that uses the longitudinal displacement along with $M_{p'}$ (1111751 inch-lb) calculated above with the previously entered axial load to calculate new unbraced lengths for both segments in the upper zone of the pile.



- Run the L-Pile® analysis with the new load case. The new unbraced lengths can be pulled from the moment graph as before. Also pull the maximum moment from the graph as well. This data can be pulled from the output by interpolation. Using the graph provides an equivalent accurate number much quicker.

$$l_{b.top} = 4.1052\,ft = 49.262 \cdot inch \tag{7-1}$$

$$l_{b.2nd} = 10.4748\,ft = 125.698 \cdot inch \tag{7-2}$$

$$M_{u.2nd} = 426.88\,inch \cdot kips \tag{7-3}$$

- Since a plastic hinge developed at the pile head, the value for K for the top segment becomes 2.1. The value for the second segment remains the same.

$$K_{top} = \begin{vmatrix} 2.1 & if \;\; M_{top} = M_{p'} \\ 1.2 & otherwise \end{vmatrix} = 2.1 \tag{7-4}$$

- With the new Unbraced lengths and the new K value, recalculate λ_{top} and λ_{2nd}.

$$\lambda_{top} = \left(\frac{K_{top} \cdot l_{b.top}}{r_y \cdot \pi} \right)^2 \cdot \frac{F_y}{E} = 0.219 \tag{7-5}$$

$$\lambda_{2nd} = \left(\frac{K_{2nd} \cdot l_{b.2nd}}{r_y \cdot \pi} \right)^2 \cdot \frac{F_y}{E} = 0.323 \tag{7-6}$$

- Now the final Nominal Structural Pile Resistance (NSPR) can be calculated for both the top and second segment.

$$P_{n.top} = \begin{vmatrix} \left(0.66^{\lambda_{top}} \cdot F_y \cdot A_s\right) & if \;\; \lambda_{top} \le 2.25 \\ \dfrac{0.88 \cdot F_y \cdot A_s}{\lambda_{top}} & otherwise \end{vmatrix} = 995.141 \cdot kips \tag{7-7}$$

- Interpolating from Table 4.5.1.5-5 (K=2.1) with l_b = 4.1052 ft $P_{n.top}$ = 994.58 kips.

$$P_{n.2nd} = \begin{vmatrix} \left(0.66^{\lambda_{2nd}} \cdot F_y \cdot A_s\right) & if \;\; \lambda_{top} \le 2.25 \\ \dfrac{0.88 \cdot F_y \cdot A_s}{\lambda_{2nd}} & otherwise \end{vmatrix} = 952.905 \cdot kips \tag{7-8}$$

- Interpolating from Table 4.5.1.5-3 (K=1.0) with l_b = 10.4748 ft $P_{n.2nd}$ = 951.89 kips.
- And the subsequent Structural Pile Resistance (SPR) for each.

$$P_{r.top} = \phi_{cu} \cdot P_{n.top} = 696.599 \cdot kips$$

$$P_{r.2nd} = \phi_{cu} \cdot P_{n.2nd} = 667.033 \cdot kips$$

- Now check the ratio of the applied load to the pile resistance to ensure the selected pile is reasonably sized as done before.

$$\frac{P_u}{P_{r.top}} = 0.598 \tag{7-9}$$

$$\frac{P_u}{P_{r.2nd}} = 0.625 \tag{7-10}$$

$$Check\left[\left(\frac{P_u}{P_{r.top}} > 0.2\right) \wedge \left(\frac{P_u}{P_{r.2nd}} > 0.2\right)\right] = \text{"OK"} \tag{7-11}$$

- Since the pile is appropriately sized, the second segment of the upper zone of the pile needs to be checked with the interaction equation of LRFD Section 6.9.2.2. It is important that this segment of the pile does not form a plastic hinge. A plastic hinge in this segment will cause the pile to fail.

$$\frac{P_u}{P_{r.2nd}} + \frac{8}{9}\cdot\left(\frac{M_{u.2nd}}{M_r}\right) = 0.792 \tag{7-12}$$

$$Check\left[\frac{P_u}{P_{r.2nd}} + \frac{8}{9}\cdot\left(\frac{M_{u.2nd}}{M_r}\right) < 1\right] = \text{"OK"} \tag{7-13}$$

- Though Shear is not a big concern for integral abutment bridges, the following shear checks are for illustrative purposes. The shear at the head of the pile can be pulled from the output L-Pile produces. Be sure to pull the shear value from the output for the load case 2.

SEGMENT OF L-PILE® OUTPUT:

```
. . .
-----------------------------------------------------------------------------
             Computed Values of Load Distribution and Deflection
                 for Lateral Loading for Load Case Number  2
-----------------------------------------------------------------------------

Pile-head boundary conditions are Displacement and Moment (BC Type 4)
Specified deflection at pile head    =        -.472400 in
Specified moment at pile head        =      1111751.000 in-lbs  <- Mp'
Specified axial load at pile head    =       416795.740 lbs

  Depth    Deflect.      Moment       Shear        Slope       Total      Soil Res.      Es*h
    X         y            M            V            S         Stress        p           F/L
   in        in         lbs-in         lbs         Rad.      lbs/in**2    lbs/in       lbs/in
-------- --------- ----------- ----------- ----------- ----------- ----------- -----------
   0.000  -.472400     1111751. -24240.4633    .0007078  55280.8639      0.0000      0.0000
   3.937  -.468016     1014489. -24179.0013    .0014838  52117.2294     31.2227    262.6493
   7.874  -.460717      916496. -23983.1504    .0021885  48929.8147     68.2695    583.3904
  11.811  -.450784      818463. -23638.0120    .0028216  45741.1152    107.0608    935.0363
. . .
```

$$\boxed{V_u = 24.2405\ kips} \tag{7-14}$$

- AASHTO LRFD does not directly address weak axis shear. This analysis will use the AISC Steel Construction Manual 13th edition (G7) to ensure the pile will not shear under the longitudinal load.

$$k_v = 1.2 \tag{7-15}$$

$$C_v = 1 \tag{7-16}$$

- Both flanges will resist shear forces

$$A_w = 2 \cdot b_f \cdot t_f = 14.884 \cdot inch^2 \tag{7-17}$$

$$V_n = 0.6 \cdot F_y \cdot A_w \cdot C_v = 446.52 \cdot kips \tag{7-18}$$

$$V_r = \phi_v \cdot V_n = 446.52 \cdot kips \tag{7-19}$$

$$Check\left(\frac{V_u}{V_r} < 1\right) = \text{"OK"} \tag{7-20}$$

$$\frac{V_u}{V_r} = 0.054 \tag{7-21}$$

- The above calculations show that shear is not a high consideration for pile design; however, it is a simple check. If L-Pile® reports high shear stresses at the pile head, this check can be done quickly. Typically, lighter piles should never encounter the shear limits due to their low resistance to longitudinal movement. The stiffer the pile, the higher the shear effects will become.
- To summarize the NSPR to this point:
  - The top segment of the pile: P_n = 995.141 kips. (7-7)
  - The second segment of the pile: P_n = 952.905 kips. (7-8)
  - The lower zone of the pile: P_n = 1090.000 kips. (6-9)
- The Nominal Axial Pile Resistance (NAPR) is only required to be R_n = 595.422 kips (4-2) for the upper zone of the pile and R_n = 833.591 kips (4-3) for the lower zone – each, well below the calculated limits. Since these values are required for the pile driving, from here on the pile capacity will be based on the NAPR. The pile design considers many levels of conservative assumptions. Driving to the pile strength will provide unnecessary capacity; increased project costs; and construction complications.
- While driving the pile, the primary forces of concern are the axial downward blow of the pile hammer and the tip resistance. The pile will be secured at the head during driving which will limit any bending. Because of this, the Nominal Pile Driving Resistance (NPDR) is based solely on the axial forces.
- The maximum stress that is permitted in the pile is:

$$\sigma_{dr} = 0.9\phi_{da} \cdot F_y = 45 \cdot ksi \tag{7-22}$$

- This translates into an ultimate maximum forces that can be applied to the pile of:

$$P_o = \sigma_{dr} \cdot A_s = 981 \cdot kips \tag{7-23}$$

- Calculate the NPDR (R_{ndr}) from the applied load divided by the resistance factor associated with the pile monitoring method.
- In this design, the pile will be bearing on rock. The chosen monitoring method is Static Analysis with pile bearing on rock.

$$\phi_{mon} = 0.45 \tag{7-24}$$

$$R_{ndr} = \frac{P_u}{\phi_{mon}} = 926.213 \cdot kips \qquad (7\text{-}25)$$

- The NPDR should not exceed the NSPR; ~~The NAPR should not exceed the NPDR;~~ nor should the NPDR exceed the maximum driving force limited by the maximum stress calculated above (7-22) [1].

$$Check\left(R_{ndr} < P_n\right) = \text{"OK"} \qquad (7\text{-}26)$$

$$Check\left(\boxed{R_{ndr} < P_o}\right) = \text{"OK"} \qquad (7\text{-}27)$$

$$\boxed{\cancel{Check\left(P_n < P_o\right) = \text{"OK"}}} \qquad (7\text{-}28)$$

- Finally check the ratios of the applied loads to the pile resistance (calculated earlier) and the NPDR to the NSPR.

$$\frac{P_u}{P_{r.2nd}} + \frac{8}{9} \cdot \left(\frac{M_{u.2nd}}{M_r}\right) = 0.792 \qquad (7\text{-}29)$$

$$Driving\_Ratio = \frac{R_{ndr}}{P_n} = 0.972 \qquad (7\text{-}30)$$

- In this example, the NPDR controls the design of the pile.

In summary:

- The selected pile is a HP 12x74
- Applied Axial Load: P_u = 416.796 kips
- The pile will form a plastic hinge at the pile head.
- The moment that will cause a plastic moment: $M_{p'}$ = 92.646 ft-kip.
- The maximum moment in the second segment: $M_{u.2nd}$ = 426.88 35.573 ft-kip.
- The NSPR: P_n = 952.905 kips.
- The NPDR: R_{ndr} = 926.213 kips.
- The NAPR for the lower zone of the pile: R_n = 833.591 kips.
- The maximum stress in the pile: σ_{dr} = 45 ksi.
- The pile design was controlled by the Pile Driving.

[1] Modified 10/01/08. The Designer need not check the relation of R_n and the R_{ndr}. Both these values represent different uses of the pile, one being the required pile resistance based on loads, and the other being the required driving resistance. Both however must be related to P_n. R_{ndr} should also be compared to P_o to ensure the nominal driving resistance does not exceed the maximum stress allowed on the pile during driving. The Designer need not compare Pn to Po.

INDEX

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Section Number ____

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_____ Typographical Error in Text

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- _____ Equation wrong

_____ Other ____

Comment or Suggestion: (Continue on back if necessary)

Name & Affiliation ____

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NOTES:

VTrans Structures Section

Montpelier, Vermont