Steel Bridge Design Handbook

CHAPTER 16
Substructure
Design

February 2022

Smarter.
Stronger.
Steel.

Foreword

The Steel Bridge Design Handbook covers a full range of topics and design examples to provide bridge engineers with the information needed to make knowledgeable decisions regarding the selection, design, fabrication, and construction of steel bridges. The Handbook has a long history, dating back to the 1970s in various forms and publications. The more recent editions of the Handbook were developed and maintained by the Federal Highway Administration (FHWA) Office of Bridges and Structures as FHWA Report No. FHWA-IF-12-052 published in November 2012, and FHWA Report No. FHWA-HIF-16-002 published in December 2015. The previous development and maintenance of the Handbook by the FHWA, their consultants, and their technical reviewers is gratefully appreciated and acknowledged.

This current edition of the Handbook is maintained by the National Steel Bridge Alliance (NSBA), a division of the American Institute of Steel Construction (AISC). This Handbook, published in 2021, has been updated and revised to be consistent with the 9th edition of the AASHTO LRFD Bridge Design Specifications which was released in 2020. The updates and revisions to various chapters and design examples have been performed, as noted, by HDR, M.A. Grubb & Associates, Don White, Ph.D., and NSBA. Furthermore, the updates and revisions have been reviewed independently by Francesco Russo, Ph.D., P.E., Brandon Chavel, Ph.D., P.E., and NSBA.

The Handbook consists of 19 chapters and 6 design examples. The chapters and design examples of the Handbook are published separately for ease of use, and available for free download at the NSBA website, www.aisc.org/nsba.

The users of the Steel Bridge Design Handbook are encouraged to submit ideas and suggestions for enhancements that can be implemented in future editions to the NSBA and AISC at solutions@aisc.org.

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3. Original Author(s) Domenic Coletti, PE (HDR) and James Sheahan, PE (HDR)	4. Revision Author(s) Domenic Coletti, PE (HDR)
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8. Abstract While the primary focus of the Steel Bridge Design Handbook is the design of steel girder superstructures, the design of the rest of the bridge is equally important. This chapter provides discusses substructure and foundation design. Many factors influence the selection of substructure and foundation materials, types, configurations, positions, and orientations. Often, existing constraints will limit the range of options and intrinsically lead the designer to only one or a few feasible solutions. This chapter provides an overview of various foundation types including both shallow foundations (such as spread footings) and deep foundations (such as those utilizing piles or drilled shafts). This chapter also presents considerations for the design and detailing of conventional, integral, and semi-integral abutments, as well as considerations for pier selection and the design and detailing of multi-column piers, single column piers, pile bents, straddle bents, and integral pier caps. Lastly, this chapter briefly discusses substructure design software, seismic design, precast substructures, soil-structure interaction, and superstructure-substructure interaction.	
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Steel Bridge Design Handbook:
Substructure Design

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1.0   INTRODUCTION

The primary focus of the Steel Bridge Design Handbook is the design of steel girder superstructures. But equally important in the overall design process is the design of substructures. This chapter will provide an overview of many issues associated with substructure and foundation design. However, this is only an overview; the reader is directed to the many other excellent references that discuss substructure and foundation design in more detail. The references mentioned at the end of this chapter constitute a brief list of some of those publications.

2.0   FACTORS INFLUENCING SUBSTRUCTURE TYPE SELECTION

Many factors influence the selection of substructure and foundation materials, types, configurations, positions, and orientations. Often, existing constraints will limit the range of options and intrinsically lead the designer to only one or a few feasible solutions. Some of these constraints are discussed below. Some are obvious while others are subtle; some are routine while others occur only rarely; but each is important and should be considered in each bridge project.

2.1   Clearance Requirements

- Vehicular Clearance Requirements – Design criteria for highway and railroad crossings typically include indication of the number, location, and alignment of lanes or tracks passing under the bridge, as well as minimum horizontal and vertical clearance requirements. These directly influence span lengths and substructure heights, and since higher bridges with longer spans often produce increased substructure loads, these are key factors affecting the choice of substructure type for these bridges.

- Navigation Clearance Requirements – Design criteria for river, harbor, or other navigable water crossings often include minimum horizontal and vertical clearance requirements for navigation. These determine span lengths and substructure heights, and since higher bridges with longer spans often produce increased substructure loads, these are key factors affecting the choice of substructure type for these bridges.

2.2   Site Conditions and Existing Constraints

- Topography and Geology – An obvious influence on the choice of substructure type is the nature of the existing terrain, both in terms of topography and geology. A bridge across a deep, rocky gorge will need different substructures than a bridge across a wide marshy swamp. Designers should consider geometry (substructure heights, span lengths, etc.), geology (which affects the selection of candidate foundation types and their effect on the overall substructure design), and constructability (physical access to build various substructure types in specific locations).

- Existing Structures, Infrastructure, Utilities – Oftentimes, especially in the more urbanized settings of many modern projects, existing structures (both above and below ground), existing roadways, existing rail or transit tracks, or existing underground or above ground utilities will limit the designer's options for substructure and foundation placement and configuration. Designers should obtain the best information available regarding these existing features before laying out substructure locations and discussing foundation and substructure types. Often a combination of existing plans, site visits, aerial survey, ground survey, and subsurface survey may be necessary to fully describe these existing conditions. Also, keep in mind structures that may not be present at the time of design, but which may be built before or during construction of the bridge.

2.3 Stakeholder Commitments

- Environmental Commitments – Transportation projects generally undergo a rigorous environmental permitting process. As part of this process, commitments are often made to various environmental agencies regarding the location and configuration of bridges and the nature of both temporary construction activities and permanent features of the project. These commitments often include very specific discussions regarding the nature of bridges crossing environmentally sensitive areas. These commitments may include limitations on span lengths, foundation types, substructure types, substructure locations, construction access and methods, etc.

- Aesthetic and Historic Commitments – The planning of transportation projects, particularly those in heavily populated or scenic areas, may include soliciting the input of stakeholder organizations and/or the general public regarding the appearance of the project's bridges and other structures. The wide range of available types, shapes, colors, and textures makes substructures an attractive candidate for aesthetic treatment. In many cases, the type and appearance of substructures are dictated by corridor aesthetic themes or owner-agency or public preferences. Designers are encouraged to embrace rather than fight this trend. Often the cost of aesthetic materials is not the driving factor in overall bridge project cost, and in many cases reasonable aesthetic details can be included in a project at little or no additional cost. The key is for the structural engineer to actively participate in the aesthetic design process so that structural and constructability considerations are appropriately addressed early on.

2.4 Constructability Considerations

- Site Access – Many factors affect site access, including topography, soil conditions, climate (weather) conditions, waterways, utilities (both above- and below-ground), existing and proposed structures, etc. Site access in turn can affect the selection of foundation types and substructure types. Designers should consider the ability of the contractor to deliver materials and operate equipment at a given site.

- Desired Construction Schedule – On many modern projects with complex construction phasing and tight schedules, the sequence and timing of substructure and foundation construction are key design criteria. Sometimes pile driving equipment physically cannot operate in a given location after a certain point in construction, e.g., due to the large vertical clearance needed for pile-driving. Sometimes waiting several days for cast-in-place concrete to cure prior to the next step in construction is not feasible. Oftentimes drilled shafts require a casing to support the open hole because another nearby structure will be built first and its loading could cause the hole to cave in. For design-build projects, detailed discussions of the proposed construction sequence with the contractor are strongly encouraged. For conventional design-bid-build projects, it may be advisable to consider several possible construction sequences and evaluate the impact of substructure type on constructability issues.

- Local Contractor Expertise – In many cases, the choices of foundation and substructure types are limited by the expertise and equipment available to local contractors. Sometimes what seems like an inefficient design may in fact be the most economical design if local contractors are equipped, experienced, and efficient at that type of work. Insights into these issues can be obtained by talking with local owner-agencies, local contractors, and other designers who have a history of design in a given locality.

2.5 Design Load Criteria

Many different types of loading are involved in substructure design, including:

- "Routine" Loading – Even so-called "routine" highway bridge design loads (such as wind loading, braking forces, centrifugal forces, etc.) can influence the choice of substructure type, particularly if other parameters such as column height amplify the effects of those forces and necessitate the choice of particular types of substructures or foundations capable of effectively resisting those force effects.

- Vessel Collision – The nature of the marine traffic may directly influence selection of substructure type. Vessel collision forces can represent a significant design loading, and can influence desirable span lengths, substructure and foundation types, and other parameters. The *AASHTO LRFD Bridge Design Specifications, 9th Edition* (referred to herein as the AASHTO LRFD BDS) [1] and *AASHTO Guide Specifications for Design for Vessel Impact, 2nd Edition* [2] offer guidance on this topic.

- Vehicular Collision – The AASHTO LRFD BDS [1] includes provisions requiring consideration of vehicular collision forces for interior bent and pier design. The design collision force is quite large (an equivalent static force of 600 kips) and can affect the choice of column size or other features of the substructure design.

- Train Collision – For bridges over railroad tracks, most railroads require that a crash wall be provided if the substructure features multicolumn bents located less than a specified minimum horizontal clearance from the centerline of track.

- Seismic Loading – For bridges located in regions of significant seismic activity (Zone 2 through 4 as defined by the AASHTO LRFD BDS [1], seismic loading can potentially have a significant effect on the substructure design and can influence the choice of substructure and foundation types. Seismic design is discussed further in Section 10.0 of this chapter.

2.6 Tolerable Movements

Excessive movements of bridge foundations, in either vertical or horizontal directions, can lead to a number of problems, including poor ride quality, undesirable appearance, damage to expansion joints, excessive cracking, or other structural integrity or serviceability problems.

Foundation movements, and the resulting structure movements, should be carefully calculated, and, perhaps more importantly, the effects of these movements on the structure should be thoroughly assessed. Tolerable movements should be estimated prior to beginning detailed design. The structural engineer should work with the geotechnical engineer regarding acceptable movements and the resulting implications in terms of both the effects on the structure and the effects on the foundations. Barker et al (*Manuals for the Design of Bridge Foundations*, NCHRP Report 343) [3] offer a good overview discussion of the issue of tolerable movements. More recently, Moon et al (*Bridge Superstructure Tolerance to Total and Differential Foundation Movements*, NCHRP Web-Only Document 245) [4], present updated guidance on this topic, geared toward implementation in the context of AASHTO LRFD BDS limit-states design.

Most of the movements discussed above are related to soil displacements caused by applied loads. Conversely, in addition to force-driven loading effects, substructures and foundations are also affected by deflection-driven loading effects, such as shrinkage, thermal expansion and contraction, etc. This is especially true when integral substructures such as integral pier caps and integral abutments are used.

For example, the use of integral abutments is often limited to superstructure units of certain length – longer lengths would result in excessive thermal movements which cannot be tolerated due to the introduction of overly large passive soil pressures resisting these movements and/or excessive bending moments in pile foundations. In other cases, excessive movements of the tops of tall piers may induce unacceptably large secondary loading effects (P-Δ effects) depending on the strength, stiffness, and height of the pier.

2.7   Scour

For bridges at stream or river crossings, scour should be investigated in detail.

Foundations should extend below the scour line. For the design flood event (typically the 50-year or 100-year design flood event), foundations should extend deep enough below the scour line to satisfy strength limit state checks. For the check flood event (typically the 500-year event, sometimes called the "superflood" event) the foundations should extend far enough below the scour line to satisfy stability criteria under extreme event limit states.

In some cases, scour prevention methods such as rip rap armoring can be used to reduce the effects of scour, especially at abutments.
The assessment of scour potential is a process which involves both the geotechnical engineer and the hydraulics engineer, in addition to the structural designer. *Hydraulic Engineering Circular (HEC) 18* [5] is a good basic resource for guidance on bridge scour, and there are numerous other report on the topic, including a series of NCHRP Reports which address various specific scour topics [6, 7, 8, and 9].

3.0   FOUNDATIONS

A wide variety of foundation types are available for use on steel girder bridges, similar to any bridge type. The choice of a preferred foundation type typically is heavily influenced by local subsurface conditions and past success with similar foundation types in the locality. In some cases, the nature of the superstructure will also influence the choice of foundation type, especially when integral substructures are used. Several of the more common foundation types are discussed in this chapter. Other types that are less frequently used and are considered as innovative foundation types, are also proposed for special circumstances. Designers are advised to involve qualified geotechnical engineers early in the bridge design process to help select the appropriate foundation type. In addition, designers are encouraged to consult one or more references on foundation design, such as references [3, 4, 10, 11, 12, 13, 14, 15, and 16].

In general, foundations can be broadly categorized as either shallow foundations (primarily spread footing foundations) or deep foundations (including driven piles, drilled shafts, and micropiles, among other types). Each category is discussed below. As the category of deep foundations is broader, with consequently more issues to consider, its discussion is further broken out in additional sections.

3.1   Shallow Foundations (Spread Footings)

3.1.1   Design Considerations

As with other types of foundations, the design of spread footings should consider both geotechnical and structural issues and criteria. The design calculations should address issues related to strength, serviceability, and movements (especially settlement). In addition, constructability issues and site-specific issues should be considered. Any of these various issues may control the design and should be evaluated.

Spread footings offer several advantages in terms of simplicity of construction, particularly when used in proper applications. The decision to use a spread footing, or any foundation for that matter, often comes down to assessing what is the most appropriate foundation choice for the given site conditions and the given structural requirements (applied loads, movements, etc.). As a general reference, Figure 1a and 1b are provided to depict a typical spread footer design for use at a stub abutment or pier, respectively

There are also many good references that discuss spread footing design, including [10, 11]. Article 10.6 of the AASHTO LRFD BDS [1] includes design provisions for spread footings.
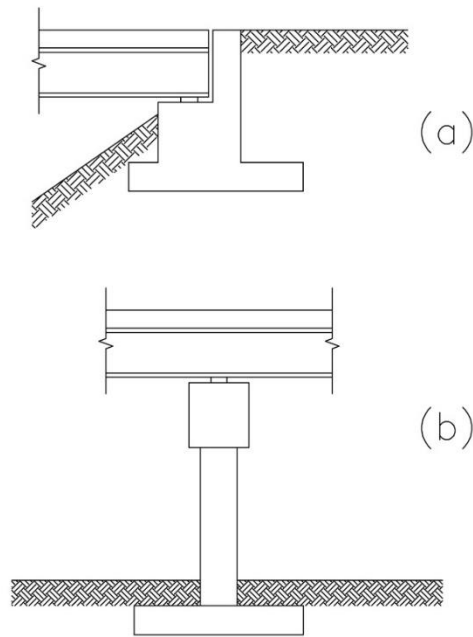
Figure 1 Typical spread footing applications: a) for a stub abutment; b) for a pier.

Spread footing design typically includes the following design checks:

- Soil/Rock Bearing Resistance – A check of the factored bearing pressure from the structure vs. the factored bearing resistance provided by the ground.

- Eccentricity (Overturning) – This check is important for abutments founded on spread footings (which are subject to lateral soil pressure applied to the abutment backwall) and for piers founded on spread footings (which may have large overturning moments, particularly if the piers are tall).

- Sliding – This check is important for abutments founded on spread footings (which are subject to lateral soil pressure applied to the abutment backwall).

- Settlement – The anticipated settlement should be determined and compared to the tolerable settlement for the structure, particularly for spread footings on soil (as opposed to spread footings on rock). References [3, 4] provide good discussion of settlement and tolerable movements.

- Horizontal Movement and Rotation – The anticipated horizontal movements (sliding displacements) and overturning rotations should be determined and compared to tolerable movements for the structure, particularly for spread footings in soil, even if the sliding and eccentricity calculations show an adequate capacity/demand ratio.

3.1.2 Detailing Considerations

In many cases, the need to size a spread footing to reduce bearing pressures, to resist sliding and overturning, and to control deflections will result in a fairly large, stout structure. The reinforcing required to resist the applied loads in structures of these proportions may be quite light. Designers are reminded to keep in mind other reinforcing requirements such as constructability, the need for reinforcing to resist temperature and shrinkage stresses, good seismic detailing practice, etc., which may end up controlling the reinforcing design. For example, larger reinforcing bars may be required simply to verify that the reinforcing cage can support its own weight prior to concrete placement.

The layout of construction joints in spread footings should be based on the evaluation of several issues. For example, the volume of concrete in each lift should be limited to control heat of hydration in larger mass concrete pours. Other practical considerations related to hydrostatic pressure of wet concrete, access for consolidation, entrapment of air in stepped concrete structures, etc., may also suggest the need for construction joints. In addition, once a reasonable construction joint layout has been determined, the reinforcing detailing should be examined to see if adding or moving reinforcing splices would improve constructability.

Keep in mind that many surfaces in spread footings will be cast against soil (or rock) or will be permanently exposed to soil. Concrete cover requirements should be adjusted accordingly. Also remember that rock cuts may not be "ideally smooth or level" and that provisions should be made to address this.

3.1.3 Construction Considerations

Construction of spread footings in soil may involve fairly easy excavation due to the soft nature of the excavated material. However, the same soft nature of the soil may also dictate the need for excavations significantly larger than the size of the spread footing, involving generous cut slopes, or temporary shoring of the excavation walls. Designers should discuss this with the project's geotechnical engineer, and check clearances to adjacent roadways, utilities, foundations, etc. Lack of adequate construction clearances may result in a design that cannot be built.

Conversely, construction of spread footings in rock may involve more difficult excavation operations, possibly necessitating specialty equipment or rock removal techniques. Consultation with experienced local contractors early in the design is advisable and can alert the designer to key issues which may have significant ramifications on their design.

In addition to providing adequate space for construction activities at the site, consideration should also be given to providing adequate access to the site. Designers should at a minimum investigate the following parameters of access/haul roads:

- Is the haul road wide enough to accommodate the anticipated equipment and trucks?

- Is the grade shallow enough for safe transit of the anticipated equipment and trucks?

- Are the anticipated equipment and trucks able to negotiate any curves in the haul road?

- Do the haul road and its subgrade have sufficient strength to accommodate the anticipated equipment and trucks?

- Is there sufficient room for required construction equipment, and is there sufficient room for the equipment to operate in the work area?

The value of consultation with experienced local contractors in assessing constructability issues early in the design process cannot be overstated.

3.1.4   Scour

As mentioned in Section 2.7 of this chapter, scour should be investigated in detail for bridges at stream or river crossings. Shallow footings such as spread footings are particularly susceptible to scour since the scour line can often extend below the bottom of the footing. In some cases, scour prevention methods such as rip rap armoring can be used to reduce the effects of scour, especially at abutments. See Section 2.7 of this chapter for selected references.

3.1.5   Deterioration

Deterioration of concrete in spread footings can be caused by any of several corrosive chemicals which are often found in soils or groundwater. Geotechnical investigations should include evaluations of the presence of these types of chemicals. If they are found to be present, appropriate protective measures should be taken. A wide range of options exists including the use of special materials (or additives to standard materials), protective surface treatments, more frequent inspection and/or maintenance intervals, and the use of conservative design assumptions where future deterioration is anticipated and the strength contribution of part of the structure is discounted.

3.1.6   Shallow Foundations (Spread Footings) in Soil

Spread footings are most often used when competent rock is found at shallow depths, but they can be successfully used for foundations on soil where scour is not a design consideration. Service level bearing resistances for spread footings in soil are generally in the range of 1 to 3 tons per square foot (tsf). This lower range of bearing resistances often limits the use of spread footings in soil to more lightly loaded structures.

A key parameter in design of spread footings in soil is the assessment of vertical and horizontal movements. Despite the relatively low applied bearing pressures associated with spread footings in soil, calculated deflections may be the controlling design parameters.

3.1.7 Shallow Foundations (Spread Footings) in Rock

As mentioned above, spread footings are most often used when competent rock is found at shallow depths. In these cases, deep foundations are often too difficult and expensive to construct, and a spread footing becomes more economically attractive.

When spread footings are founded on rock, some of the concerns and issues associated with spread footings founded on soil are mitigated, but other issues arise.

Service level bearing resistances for spread footings in rock are generally in the range of 5 to 10 tsf or higher. These higher bearing pressures can potentially allow much more heavily loaded structures to be founded on spread footings.

However, the sizing of a spread footing is dependent on more than just an assessment of the bearing resistance. Sliding and eccentricity (overturning) checks should also be performed, especially for abutments, which have significant lateral loading from soil pressure, but also for piers which can have high overturning moments.

Settlement of spread footings in rock is of less concern than for spread footings in soil but should still be investigated at least in a cursory manner and compared to tolerable movements.

Construction of spread footings in rock can be expensive if a significant amount of rock excavation is required. In addition, as mentioned above, designers should also note that the resulting cut rock surface may not be ideally "smooth and level" and should adjust their detailing accordingly to allow for more generous construction tolerances. Alternatively, a slab of lean (unreinforced) concrete, sometimes called a "working slab" or a "mud slab," is often placed in the bottom of the excavation to provide a more level and uniform bearing surface.

3.2 Deep Foundations – General Design Considerations

3.2.1 Geotechnical Design Considerations

Deep foundations provide support for substructures in ways that are fundamentally different from spread footings. There are several different types of deep foundations; in broad terms the most common categories include driven piles, micropiles, and drilled shafts (sometimes also called drilled piers or drilled caissons). For the purposes of most of the discussions in this chapter, driven piles and micropiles will collectively be referred to as "piles."

Deep foundations such as piles and drilled shafts are typically long, column-type elements which develop vertical capacity by means of end-bearing in a relatively deep bearing stratum, side friction through part or all of their depth, or a combination of both. Vertical piles and drilled shafts develop lateral capacity by means of embedment of the pile or shaft into the soil to generate lateral resistance of the soil against the side of the pile or shaft. Battered piles develop lateral capacity through the horizontal component of the axial resistance in the battered pile,

sometimes in combination with embedment of the pile into the soil to generate lateral resistance of the soil against the side of the pile. Piles and drilled shafts typically achieve overturning capacity by means of either group action, where the overturning moments are resolved into axial force couples distributed among grouped piles or drilled shafts, or on an individual basis acting as flexural elements embedded in the soil.

Resistance to Vertical Loads

For end-bearing piles or drilled shafts, the vertical capacity arises from the bearing of the end of the foundation element on a competent stratum of soil or rock and is thus independent of the length of the pile or drilled shaft. Typically, the main design parameters are the bearing resistance of the soil or rock, the axial capacity of the structure element (pile or shaft), and the end-bearing area. Note, however, that a minimum length of embedment of the pile or drilled shaft into the ground is also usually required for other reasons, as will be discussed below.

For side friction piles or drilled shafts, the vertical capacity is derived from adhesion or friction between the sides of the pile or drilled shaft and the surrounding soil or rock. The typical design parameters are the length of the pile or drilled shaft in the stratum providing frictional resistance, the perimeter of the pile or drilled shaft, and the frictional capacity of the soil or rock. Typically, the development of the frictional resistance requires some amount of vertical movement (displacement) of the pile relative to the surrounding soil to mobilize the side friction resistance of the soil.

Note that the side friction capacity of closely spaced piles or drilled shafts can be adversely affected by so-called "group effects." In simple terms, a closely spaced group of piles or drilled shafts can begin to act as a single entity with an overall perimeter measured around the outside of the group (due to the overlapping zones of soil movement around each pile which reduce the soil's strength) rather than as a collection of individual element perimeters. The reduction in side friction capacity can generally be mitigated by designing the foundation with a sufficient pile or drilled shaft spacing.

Piles and drilled shafts can develop vertical capacity by means of both side friction and end-bearing acting simultaneously, depending on the specific subsurface conditions. There are no easy and consistent rules for the calculation of vertical capacity – designs should be addressed on a case-by-case basis by qualified geotechnical engineers using site-specific data.

Note also that the effects of downdrag on the axial resistance of piles and drilled shafts should also be considered as appropriate. Downdrag occurs when the soil surrounding a pile or drilled shaft is subject to settlement after construction of the foundation, producing a side-friction force on the foundation in the downward direction.

Resistance to Lateral Loads

The determination of the lateral capacity of piles and drilled shafts is a more complex problem than the determination of vertical capacity. Several reports address this topic, including the

relatively recent and comprehensive, *Design, Analysis, and Testing of Laterally Loaded Deep Foundations that Support Transportation Facilities, Report FHWA-HIF-18-031* [17].

Typically, the lateral capacity should be calculated by means of a lateral pile analysis, usually facilitated by computer modeling. In determining lateral resistance of deep foundations, performance is related to lateral displacement and overall stability rather than strength in the strict sense. The limiting parameter in calculating the geotechnical lateral capacity of a pile or drilled shaft is often the tolerable movement which the structure can sustain.

A second, but equally important, parameter in evaluating lateral capacity of piles and drilled shafts is the overall stability of the pile as a function of its embedment into the soil. The pile or drilled shaft should have sufficient embedment in the soil to resist global rotation. In other words, the pile or drilled shaft should have sufficient embedment such that the lower end remains fixed against both translation and rotation, so that the pile or shaft will bend, rather than rotating as a rigid body, when subject to lateral loading; this is sometimes called having sufficient embedment to achieve "fixity." The associated so-called "depth to fixity" should not be confused with the "equivalent point of fixity" sometimes used as part of a simplified representation of the response of a foundation to lateral loading (discussed in Section 3.2.2 of this chapter).

Global overturning moments applied to foundations consisting of a group of piles or drilled shafts are often resisted as force couples. For example, a common foundation design involves the support of each column in a bent on a group of piles connected to the column by means of a relatively rigid footing, typically called a "pile footing" or a "pile cap." In such a design, the overturning moment is often resolved into vertical force couples and the resulting upward and downward forces on individual piles or drilled shafts are combined with the loads caused by the overall vertical load in the column (distributed among the piles or drilled shafts). In this case, the geotechnical capacity of the foundation in resisting the overturning moments is simply a function of the axial capacity of the individual piles or drilled shafts. This is a simplified, and generally conservative, design approach. When additional refinement is warranted, or when the soil conditions indicate the need, a more sophisticated soil-structure interaction analysis can be performed which considers both the ability of individual foundation elements to resist some of the overturning moment via bending, in combination with the axial force couple effect.

Conversely, in the case of "pile bents" the piles or drilled shafts continue uninterrupted above finished ground to the bent cap. In such cases, the piles will directly carry applied moments via bending in the piles. In these types of design, some form of soil-structure interaction analysis is required; at a minimum at least a single pile soil-structure interaction analysis is needed to evaluate the response of the pile or drilled shaft to lateral loading.

In either case, the lateral analysis should consider the horizontal loads on the piles or drilled shafts and evaluate the resulting horizontal movements at the tops of the foundation elements.

3.2.2 Structural Design Considerations

In addition to evaluating its geotechnical capacity, the pile or drilled shaft itself should be evaluated for its structural capacity. Depending on the nature of the loads and the nature of the structural configuration of the foundation, this might involve evaluating axial capacity, bending capacity, shear capacity, or a combination thereof. The required structural design calculations are typically straightforward and are similar to the corresponding design calculations required for columns.

However, the determination of boundary conditions for the structural analysis of deep foundations can be complicated. The boundary conditions for lateral support and stability are primarily a function of the subgrade modulus of the surrounding soil, but the restraint provided to the top of the foundation element by the rest of the structure is also important. These boundary conditions can be quantified by means of either a simplified or a more rigorous analysis.

For a simplified analysis, a pile or drilled shaft can be modeled as a column fixed at its base, neglecting any other lateral support along its length, and with boundary conditions at the top of the pile or drilled shaft based on the structural configuration. This is a modeling simplification which neglects the lateral support provided by the soil over the buried depth of the pile or drilled shaft. This theoretical fixed base is called the "equivalent point of fixity" for simplified structural analysis. The assumption is that the moments in the pile or drilled shaft at this theoretical fixed base (caused by lateral loading) approximately match the maximum moments in the pile or drilled shaft that would be determined by a more refined soil-structure interaction analysis.

The depth of the pile or drilled shaft to the assumed fixed base is known as the depth to the "equivalent point of fixity" and should not be confused with the geotechnical "depth to fixity." As mentioned in Section 3.2.1 of this chapter, piles and drilled shafts are typically installed with sufficient embedment into the soil to develop fixity in the soil. This is not the same as the structural "equivalent point of fixity" described in the preceding paragraph.

The depth to the structural "equivalent point of fixity" can be estimated by means of rules of thumb or approximate equations presented in C10.7.3.13.4 of the AASHTO LRFD BDS [1] (for approximate or preliminary analysis) or by means of a more rigorous laterally loaded pile analysis. For a more rigorous analysis, the pile or drilled shaft is modeled with nonlinear lateral spring supports where the spring constants are based on the subgrade modulus of the surrounding soil layers. This type of analysis can be done either using specialty soil-structure interaction modeling software or using general finite element analysis software.

Depending on the geotechnical engineer's recommendation or routine practice of the local owner-agency, the structural "equivalent point of fixity" is typically assumed to be either at the highest point of moment inflection or the highest point of zero horizontal deflection for a given lateral loading.

Designers are also encouraged to consider the boundary conditions at the top of the piles or drilled shafts. The nature of the substructure and its connection to the foundation, as well as the

nature of the superstructure and its connection to the substructure, directly affect the support offered to the foundation elements. For example, a fully integral connection to the superstructure can add significant support to a substructure/foundation system. There can be a temptation to over-simplify the analysis, which should be avoided. Depending on the nature of the loading, some simplifying assumptions may ultimately prove to be unconservative.

When the soil conditions, the configuration of the structure, or the nature of the loading suggest that a more refined analysis may be warranted, a comprehensive soil-structure interaction analysis can be undertaken. This type of analysis can be performed by commercial soil-structure interaction software capable of modeling single or multiple piles or drilled shafts and the substructure that they are supporting, or in general finite element analysis software. In either case, coordination with the project geotechnical engineer is essential in determining how to correctly represent the response of the soil to lateral loading and in sanity-checking the analytical results for reasonableness.

3.2.3 Choosing Between Piles and Drilled Shafts

There are myriad considerations in choosing between using pile foundations and drilled shaft foundations, including:

- Local Subsurface Conditions: Piles and drilled shafts each have advantages and disadvantages based on the nature of the subsurface conditions. Often these conditions will clearly suggest one over the other.

- Local Economic Conditions: Local contractors often have a preference for either piles or drilled shafts based on their experience with one or the other, the availability of laborers with the required specific skills, the availability and cost of specialty equipment, the ability to self-perform the work versus having to engage a specialty subcontractor, etc.

- Structural Considerations: Depending on the nature of the loading and the configuration of the rest of the structure, either piles or drilled shafts may be a better choice. For example, individual larger diameter drilled shafts may be more efficient at carrying lateral loads and moments than individual, smaller-size, driven piles.

- Constructability Considerations: Either piles or drilled shafts may be a better choice depending on any of a number of constructability issues, including the nature of the subsurface conditions, whether the pier is on land, in shallow water, or in deep water, equipment access, topography, overhead clearance, or the configuration of the rest of the structure.

- Environmental Considerations: Some foundation types and some foundation construction methods are more damaging to the environment than others. Close coordination with environmental permitting specialists may be necessary to evaluate the environmental impacts, and environmental commitments may influence the selection of the preferred foundation type and construction method.

4.0   DEEP FOUNDATIONS – PILES

One broad class of deep foundations are driven piles. Driven piles are predominantly characterized by the fact that driving operations represent either a large part or the entirety of the field construction operations. Piles are also typically, but not always, characterized by relatively slender cross-sectional dimensions compared to drilled shafts. Report FHWA-NHI-16, *Design and Construction of Driven Pile Foundations* [12] provides detailed discussions of driven pile foundations. In addition, Figure 2a and 2b, depict the application of driven piles at a stub abutment and under a pier, respectively.
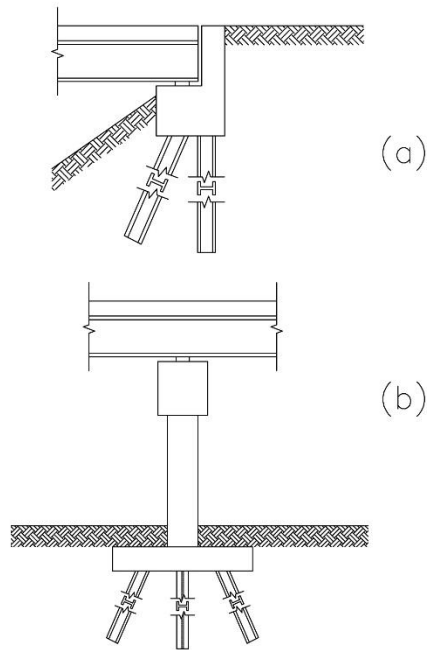
Figure 2 Driven pile applications: a) under a stub abutment; b) under a pier.

4.1   Pile Types

Driven piles can be categorized as either displacement piles, with either a solid section or a closed end, or non-displacement piles which have an open end with a relatively small cross-section. There is a very wide range of driven pile types available to the designer, including:

- Driven Steel H-Piles – These are typically AISC HP sections, depicted in Figure 3, which are open H-shaped rolled steel members. In some cases, "driving plates" (flat plates welded across the cross section at the lower tip of the pile) are provided to increase end-bearing area. In other cases, "pile tips" (serrated end fittings) are provided to allow the lower tip of a pile to "bite" into the bearing stratum or to reinforce the pile tip when driving through a hard stratum. Figure 3 depicts the foundation consisting of two rows of driven steel H-piles supporting a conventional abutment that will be located behind an

MSE retaining wall; see Figure 12 for a later photo after MSE walls and the abutment cap and backwall were built

Figure 3 Driven steel H-piles.

- Driven Steel Pipe Piles – These are hollow circular steel pipe sections. In some cases, driving plates are provided to increase end-bearing area. Pipe piles are sometimes selected over H-piles due to the greater structural capacity of the pipe section (e.g., greater bending stiffness, greater buckling capacity, greater moment capacity, etc.), while H-piles may be selected in some cases (such as integral abutments) based on their lesser bending stiffness. To further increase capacity, circular steel pipe sections can be filled with concrete (referred to as concrete filled steel tubes (CFSTs) in order to generate higher axial and bending capacities due to the composite action between the steel and concrete.

- Driven Precast Concrete Piles – These are generally precast concrete square, octagonal, or round cross section shapes, as shown in Figure 4. Precast piles are often fabricated with a central void to reduce their weight. Precast concrete piles are also often prestressed to increase their bending capacity and their resistance to tension stresses during driving.

Figure 4 Driving of a precast concrete pile.

- Timber Piles – Timber piles may seem like "obsolete" technology, but they still are viable in certain situations. Due to their relatively low structural capacity, the use of timber piles is usually limited to lightly loaded structures. But when used in proper applications, timber piles can provide advantages over other pile types. One example of a good timber pile application would be the foundation of a lightly loaded pedestrian bridge in a remote area with easy access to plentiful supplies of high-quality treated timber, but where delivery of other materials may be relatively expensive.

4.2   Factors Affecting the Choice of Pile Type

The choice of pile type is influenced by many considerations. Some examples are provided below, but designers are reminded that foundation type selection can be a complex process with many competing issues. Some examples are:

- Precast concrete piles are more often used in situations where steel pipes would be subject to severe corrosion. However, solid precast concrete piles (or piles with driving plates) may not be suitable in dense soils where driving of solid or closed end piles would be difficult or impossible.

- Steel pipe piles and precast concrete piles may be preferred over steel H-piles in situations involving long exposed, unbraced pile lengths where the higher buckling and flexural capacity are needed.

- Steel piles may be preferred over concrete piles when the required pile lengths are long enough that piles will need to be spliced. Splicing of precast concrete piles can be costly and difficult, especially if a full moment splice is desired. On the other hand, it is relatively easy to achieve a full moment capacity splice of a steel pile by means of field welding.

- Timber pile use is generally limited to lightly loaded structures, particularly when site access is restricted and when high quality timber is readily available locally.

- Steel H-piles are attractive when there is a high end-bearing capacity in soil or rock such that only a small cross-sectional area is needed in the pile to develop the required end-bearing resistance.

- Piles with larger cross sections and/or perimeters are attractive in cases where side friction is the predominant source of vertical capacity due to their greater perimeter (greater area available for generating side friction resistance).

4.3 Specific Design Considerations for Piles

In addition to the general design considerations for deep foundations listed above, some considerations specific to the design of piles are listed below.

Designers are reminded that driven piles are generally relatively slender members. Their structural analysis should consider careful evaluation of their buckling and bending capacity, and need to include consideration of second-order slenderness effects (P-Δ effects), particularly in pile bent applications (which will be discussed in more detail later in this chapter).

In many bridge foundation applications, piles are used in groups. Designers are cautioned to be aware of pile spacing limitations and overall pile group geometry and to be aware of the potential for such adverse situations as:

- Reduction in lateral and/or vertical capacity due to group effects.

- Possible uplift situations due to overturning moments.

- Possible interferences between adjacent piles when one or more piles are battered.

Care should be taken when using battered (or "brace") piles. Battered piles can offer a simple solution for providing lateral capacity to pile foundations if that lateral capacity is carefully evaluated, the increase in axial load due to the axial component of the lateral load on the battered pile is considered, and the potential for pile interferences is checked.

Also, designers are advised battering too many piles can produce adverse performance. In certain cases, particularly in abutments, battering too many piles in only one direction may potentially result in a situation where the abutment deflects into the retained fill. In this situation, the horizontal component of the battered pile loads generates a lateral load on the fill; this load is resisted by passive pressure, but only after sufficient deformation occurs in the soil to generate the needed passive resistance.

Designers are also advised to be aware of construction tolerances for out-of-plumbness and for top of pile out-of-position. These construction tolerances lead to eccentricities in the application

of axial load which should be considered in the design of the pile. Keep in mind that the effects of out-of-plumbness and top of pile out of position can potentially be additive, depending on how the project specifications are written and on the contractor's ability to control the pile during driving. For example, a pile may be significantly out of position at its lower end while still meeting the out-of-plumbness requirement, and simultaneously the top of the pile may be out of position in the opposite direction while still meeting the out-of-position requirements. In this case, the net effective eccentricity is a combination of both effects.

Article 10.7 of the AASHTO LRFD BDS [1] includes design provisions for driven piles.

4.4 Detailing Considerations for Piles

- Pile Spacing – As previously mentioned, pile spacing can affect both the axial and lateral geotechnical capacity of piles as well as the load distribution in piles.

- Footing or Pile Cap Embedment – Piles should be adequately embedded into pile caps or footings. The determination of adequate embedment should consider:
  a. the capacity to transfer pile axial load via end-bearing and side friction in the concrete;
  b. the capacity to transfer pile bending moments via compression block force couples as described on page 11 of reference [18];
  c. the capacity to transfer pile shear forces via bearing of the embedded pile on the surrounding concrete; and
  d. the capacity to develop pullout (tension) resistance or accommodate uplift anchors in the case of piles subject to uplift.

- Footing Reinforcement – Footing reinforcement should be designed following standard footing design guidelines which can be found in many references, including the AASHTO LRFD BDS [1]. Strut-and-tie modeling may also be an appropriate method of analysis of footings.

4.5 Miscellaneous Considerations for Piles

4.5.1 Construction

Pile construction considerations are myriad. Some considerations include:

- Drivability – Depending on the subsurface conditions, pile driving may be difficult or physically impossible. In some cases, predrilling might be a solution; if so, the designer and the geotechnical engineer should discuss appropriate backfilling measures.

- Access – Pile driving equipment is rather large. Sufficient access for the equipment should be investigated. Moreover, sufficient horizontal and especially vertical clearance is required for the equipment to operate properly.

- Subsurface Obstructions – Hitting a boulder or an underground utility can ruin a pile driving contractor's day; sufficient utility survey and sufficient geotechnical field investigations are usually worth their costs. In extreme cases, such as hitting an underground gas line, the results of unforeseen interferences can be catastrophic.

- Need for Pile Splicing – Long piles will require splices. Depending on the splice requirements, the need for splicing long piles may preclude certain pile types.

- Need for Pile Tips – When driving piles to bearing on rock, it may be advisable or even mandated that pile tips, shown in Figure 5, be used to avoid problems with the pile "skipping" or "skidding" on the rock stratum that may not be level.

Figure 5 Example of pile tips.

4.5.2   Scour

Scour around pile foundations, particularly around groups of piles, can be severe. The pile causes a disruption in flow which produces eddies (turbulence) that generate local (pier) scour. Scour holes around piles reduce the pile's lateral support and side friction capacity. Both the geotechnical and structural analysis of the pile foundations should include consideration of scour. Scour effects should be quantified by a detailed scour analysis [5]. See Section 3.1.4 of this chapter for more discussion of scour.

4.5.3   Deterioration

As mentioned above for spread footings, deterioration of concrete and steel can be caused by any of a variety of corrosive chemicals which are often found in soils or groundwater. Geotechnical investigations should include evaluations of the presence of these types of chemicals. If they are found to be present, appropriate protective measures should be taken. A wide range of options exists, including the use of special materials (or additives to standard materials), protective surface treatments, more frequent inspection and/or maintenance intervals, and the use of conservative design assumptions where future deterioration is anticipated and the contribution of part of the structure is discounted.

4.5.4 Verification of Resistance

Pile resistance is most often verified by means of simple pile hammer blow count evaluations; as part of routine recordkeeping, the number of blows of known force (known weight of hammer and known hammer stroke) required to achieve a specified movement of the pile is recorded. A driving formula (FHWA Gates formula, for example) can be used to evaluate the resistance of the pile. This is a simple, but effective method. More sophisticated methods are sometimes required by contract specifications, usually as a means of spot-checking pile capacity. One such method is the use of a Dynamic Pile Tests with signal matching (Pile Driving Analyzer (PDA), for example). Dynamic pile testing uses strain gages and acceleration transducers to evaluate bearing resistance, pile integrity, and driving stresses. In some cases, a static or Statnamic load test is required to demonstrate adequate axial or lateral resistance of a pile. A static load test involves loading the pile with a force generated by a jack pushing against a reaction frame. A Statnamic test involves generating gas pressure within a sealed chamber located on top of test pile, with a reaction mass above the chamber; the increase in pressure inside the chamber creates equal and opposite forces on the reaction mass and the test pile [12].

5.0   DEEP FOUNDATIONS – DRILLED SHAFTS

Drilled shafts represent a somewhat narrower range of deep foundation types. Drilled shafts can generally be characterized by a few common features: a relatively deep, round hole excavated in the ground and backfilled with reinforced concrete. For clarity, Figure 6a and 6b show the applications of drilled shafts at a stub abutment and pier, respectively. A good, comprehensive discussion of drilled shaft design and construction can be found in Report FHWA-NHI-18-024, *Drilled Shafts: Construction Procedures and Design Methods* [13].
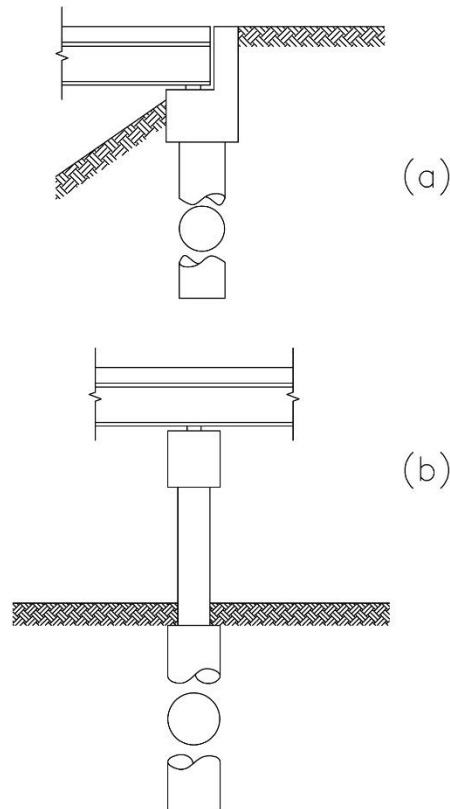
Figure 6 Drilled shaft applications: a) for a conventional stub abutment: b) for a pier.

Drilled shafts are sometimes categorized by the source of their vertical capacity as either end-bearing drilled shafts, side friction drilled shafts, or a combination of the two.

There are several other features which vary from one drilled shaft application to the next, including:

- Casing – When shafts are drilled through soils subject to caving, steel casing, shown in Figure 7 below, is often used to keep the hole open until concrete is placed. In some situations, the casing is then extracted; in other situations, it is left in place.

- Bottom Configuration – In some instances, the bottom of a drilled shaft is widened to increase the end-bearing area. These are called "belled" drilled shafts in reference to the bell shape at the tip of the shaft. This is only possible in soil.

- Rock Sockets – In some cases where drilled shafts are installed in rock with soil overburden, the shafts are drilled a short depth into rock to obtain reliable end-bearing or to achieve lateral fixity. In some cases, the diameter of these rock sockets is slightly less than that of the rest of the drilled shaft.

- Concrete Placement Method – There are several options for placing concrete in drilled shafts. The choice of which option to use is typically dependent on the conditions in the shaft at time of placement, the presence of casing, and/or the presence of water or slurry in the shaft. Clean, dry shafts can have their concrete dropped, at least up to a specified maximum drop height. If there is ground water in the shaft, concrete is sometimes placed under water using a tremie tube (kept embedded in the wet concrete). In other cases, a slurry is used to displace the water and the tremie-placed concrete then displaces the slurry. The use of the slurry-displacement method offers the advantage of avoiding the need for casing since the slurry can be used to prevent cave-in of unstable soils. However, the presence of the slurry prevents the inspection of the bottom of the shaft excavation before concrete placement, and disposal of the slurry after concrete placement can add cost or complexity to the construction effort.

5.1   Factors Affecting Choice of Drilled Shaft Type

The main factors affecting the choice of features in a drilled shaft are typically the subsurface conditions and constructability issues. The presence of groundwater and/or unstable, caving soils will have a direct impact on whether the shaft will need to be cased and what type of concrete placement method to use.

The use of belled drilled shafts is not very common anymore, primarily due to the difficulties associated with their construction and with ensuring a clean bottom for end-bearing.

The need for rock sockets is typically determined based on the required vertical capacity and lateral fixity and how these can be achieved. If sufficient vertical capacity and lateral fixity can be achieved by means of side friction through soil only, rock sockets may not be required.

5.2   Specific Design Considerations for Drilled Shafts

In addition to the general design considerations for deep foundations listed above, designers are reminded that while drilled shafts are generally relatively stocky members, their structural analysis should consider careful evaluation of their axial and bending capacity, usually by means of an axial-bending interaction analysis. Drilled shaft structural design is essentially identical to regular reinforced concrete column design and should be approached in that manner.

Depending on the structural configuration, the analysis may need to include consideration of second-order slenderness effects (P-Δ effects). In many cases on bridges with multicolumn bents, a single drilled shaft is provided for each column with no intermediate pile cap. In those cases, the column is literally an extension of the drilled shaft and, if the column is fairly tall and slender, second-order slenderness effects may become significant. The AASHTO LRFD BDS [1] (Article 5.6.4.3) allows that slenderness effects can be neglected in columns when KL/r is less than 22, provides an approximate method for evaluating moment magnification in columns when KL/r is between 22 and 100, and suggests that a second-order analysis (a P-Δ analysis) be used to evaluate moments in columns when KL/r exceeds 100.

Figure 7 Single round column with an oversized, steel cased, drilled shaft.

In other applications, drilled shafts may be used in groups connected by a pile cap. Designers are cautioned to be aware of drilled shaft spacing limitations and overall drilled shaft group geometry and to watch out for the potential for such adverse situations as:

- Reduction in lateral and/or vertical capacity due to group effects.

- Possible uplift situations due to overturning moments.

Drilled shafts are seldom, if ever, installed in battered configurations to address lateral loading. Instead, lateral load is applied to the plumb drilled shaft and is carried via the bending capacity of the shaft until the load is transferred to the soil through the significant projected lateral bearing area of the shaft; frame action can be considered in the case of groups of shafts connected by a common pile cap or bent cap. Note that this load transfer mechanism contributes to the previously mentioned design moments in drilled shafts, and these moments should be considered in the structural design of the drilled shaft.

As is the case for other types of foundation designs, in addition to checking structural and geotechnical capacities, settlement and horizontal movements should be calculated for drilled shafts and compared to tolerable movements.

Be aware that some vertical movement may be required to generate end-bearing capacity in drilled shafts. One rule of thumb is that vertical movement of about 5% of the shaft diameter is required to generate full end-bearing capacity. In some cases, geotechnical engineers will limit the use of the end-bearing capacity to as little as 25% of the full end-bearing capacity in order to limit the vertical movement of the shaft.

Similarly, a common rule of thumb is that about ½" of vertical movement is required to mobilize side friction capacity.

Article 10.8 of the AASHTO LRFD BDS [1] includes design provisions for drilled shafts.

5.3   Detailing Considerations for Drilled Shafts

- Drilled Shaft Spacing – As previously mentioned, drilled shaft spacing can affect the geotechnical capacity of the shafts (both the axial and lateral capacity) as well as the load distribution in the shafts.

- Casing – As mentioned above, casing may be required when drilled shafts are constructed in unstable soils which are subject to caving. Casing may also be required for underwater drilled shaft construction such as in creek and river crossing bridges. The casing is typically advanced as the shaft is drilled, although sometimes in weak soils the casing may be driven into position prior to drilling. The casing is typically large diameter steel pipe, with wall thickness designed to resist soil pressure or hydrostatic pressure and also to resist vertical loading either from installation loads or from loads applied by equipment or platforms which may be mounted on the casing. Casing may be left in place or may be removed, depending on a variety of considerations, including ease of removal.

- Drilled Shaft Reinforcement, General – Drilled shafts are typically reinforced in a manner similar to reinforced concrete columns. Vertical reinforcing comprises the primary steel. Spiral reinforcing is often used for confinement, although circular hoops have also been used. The full reinforcing cage is typically assembled and lowered into the shaft as a unit.

- Drilled Shaft Reinforcement, Spacing – Reinforcement spacing should be carefully evaluated and limited. Concrete placement in drilled shafts is different from that for columns. Consolidation is achieved more by natural flow due to the limited ability to vibrate the concrete within a drilled shaft. Overly tight reinforcement spacing (either in the vertical reinforcement or the confinement steel) can cause problems with consolidation, problems which are hard to identify and rectify due to lack of access. In addition, in deep drilled shafts, reinforcement may need to be spliced; lap splicing may be impractical if the main reinforcement spacing is already tight. The use of bundled primary reinforcement can be considered, as well as the use of self-consolidating concrete.

- Detailing for Inspection – Cross-hole sonic logging (CSL) tubes are often required and may add to reinforcing congestion problems. At least two CSL tubes are required in a drilled shaft to perform CSL testing; typically, four or more are provided. CSL tubes are galvanized steel or PVC pipes, typically 1.5" to 2" in diameter, placed around the perimeter of the reinforcing cage. The CSL process is further explained in the next section.

- Footing or Bent Cap Attachment – Drilled shafts are cast in place concrete structures, so they do not need embedment into a footing or bent cap per se, but the reinforcement within a drilled shaft is typically projected into the footing or bent cap, usually a full development length. Some owner-agencies prefer to provide hooks on the projecting reinforcement. Reinforcement projections which provide less than full development of the bars can be used if shown adequate by detailed design calculations, but their use is not encouraged.

- Footing Reinforcement – Footing reinforcement should be designed following standard footing design guidelines which can be found in many references, such as AASHTO LRFD BDS [1].

5.4 Miscellaneous Considerations for Drilled Shafts

5.4.1 Construction

Drilled shaft construction considerations are myriad. Some considerations include:

- Drilling – Drilling of drilled shafts is generally a large-scale construction activity, as depicted in Figure 8, and requires careful planning on the part of the contractor. Designers should be aware of key aspects of drilled shaft construction to avoid designs and details that restrict the contractor's options. For example, spoils should be collected and disposed of in an acceptable manner; environmental commitments often limit the options for spoil disposal.

- Access – Drilled shaft drill rigs are not small, and sufficient horizontal and vertical clearance is required for the equipment to operate properly. The structural engineer should coordinate with the geotechnical engineer and/or potential foundation contractors to determine if enough space is available for construction of drilled shafts. Keep in mind that a drill rig is not the only piece of equipment required. Cranes are needed to lift and place reinforcing cages in the open drilled shafts and concrete delivery trucks and pumpers are needed to deliver and place concrete in the shafts, often using tremie concrete placement methods.

- Subsurface Obstructions – Hitting a boulder or an underground utility can cause delays, equipment damage, or other problems. In extreme cases, such as hitting an underground electric or gas line, the consequences of unforeseen subsurface conflicts can be

catastrophic. Detailed utility surveys and geotechnical field investigations are typically well worth their cost.

Figure 8 Drilled shaft drilling rig.

5.4.2   Scour

Scour around drilled shaft foundations, particularly around groups of drilled shafts, can be severe, as is detailed in Figure 9**Error! Reference source not found.**. The drilled shaft causes a disruption in flow which produces eddies that generate local (pier) scour. Scour holes around drilled shafts reduce the shaft's lateral support and side friction capacity. Both the geotechnical and structural analysis of the drilled shafts should include consideration of scour. Scour effects should be quantified by a detailed scour analysis [5]. See Section 3.1.4 of this chapter for more discussion of scour.

Figure 9 A drilled shaft that has experienced severe scour.

5.4.3   Deterioration

As mentioned above for spread footings, deterioration of concrete can be caused by any of several corrosive chemicals which are periodically found in soils or groundwater. Geotechnical investigations should include evaluations of the presence of these types of chemicals. If they are found to be present, appropriate protective measures should be taken. A wide range of options exists, including the use of special materials (or additives to standard materials), protective surface treatments, more frequent inspection and/or maintenance intervals, and the use of conservative design assumptions where future deterioration is anticipated and the contribution of part of the structure is discounted. The AASHTO *Guide Specification for Service Life Design of Highway Bridges* [19] provides extensive discussion of concrete deterioration and protection strategies.

5.4.4   Inspection and Verification of Geotechnical Resistance

Drilled shaft capacity is sometimes directly verified by means of static load testing or Osterberg Load Cell (O-cell) testing. The O-cell is a specially designed hydraulic jack lowered to the base of the shaft hole with the reinforcing cage. After concrete placement and curing, the O-cell is pressurized and causes an upward force on the shaft and a downward force on the foundation material. Side friction and end-bearing resistances are measured and compared to the required design values. Reference [20] discusses the O-cell in detail. Typically, not all shafts are tested, but rather a single shaft or a small number of shafts are tested to establish capacity parameters as part of a test shaft program.

For smaller, more routine projects and/or during production, reliance is often placed on measurements of the shaft diameter, depth, characteristics of the material drilled through, and measurements of the volume of concrete placed and concrete strength. For drilled shafts designed primarily as end-bearing foundations, the bottom of the shaft hole should be inspected

visually. This can sometimes be done by direct visual inspection if the shaft is fairly shallow, dry, and large enough diameter. In other cases, a Shaft Inspection Device (SID) is used; the SID is a remote camera system allowing inspection of the bottom of deep shafts.

5.4.5   Inspection and Verification of Structural Resistance

The integrity of concrete placed in drilled shafts can be verified using cross-hole sonic logging (CSL) testing. For this testing, galvanized steel or PVC pipes are installed around the perimeter of the shaft prior to concrete placement and filled with water. Ultrasonic probes are lowered into the tubes and measurements taken. Voids in the concrete as small as 2.5" can be identified. Thermal Integrity Profiling (TIP) can also be used to evaluate the integrity of the concrete. TIP measures the heat as the concrete cures as a means of determining the quality of the concrete.

6.0 DEEP FOUNDATIONS – MICROPILES

Micropiles have been used extensively in Europe for many years. Their use as bridge foundation elements in the US did not start until later but has gained popularity in recent years. Micropiles typically consist of a relatively small diameter (9 to 12 inches) steel casing filled with grout, often with reinforcing provided in the grout. Typically, a micropile is constructed by drilling and advancing the steel casing to a prescribed depth in an identified bearing stratum, retracting the drill, placing the reinforcement and grout, and then partially retracting the casing and injecting additional grout under pressure as needed to provide end-bearing and side friction capacity in the bearing stratum. The casing is typically provided in segments that are connected together using threaded joints to produce the full casing length needed. *Report FHWA-NHI-05-039, Micropile Design and Construction* [15] provides a comprehensive discussion of micropiles, and Article 10.9 of the AASHTO LRFD BDS [1] includes design provisions for micropiles.

Micropiles offer specific advantages in certain applications. They can be installed when access is restricted, due to the small size of the equipment and materials used in their construction. They can be used in a wide variety of subsurface conditions and can resist significant axial loads. But, micropiles generally offer limited resistance to lateral loading and bending, partly due to their relatively small cross-sectional dimensions and properties, and partly due to the weak resistance to bending moments at the threaded joints connecting the casing segments.

As with other types of foundation designs, close coordination between the structural engineer and the geotechnical engineer is needed.

7.0   ABUTMENTS (END BENTS)

Abutments (a.k.a. end bents) support the superstructure at the ends of a bridge. Typically, abutments resist not only loads from the superstructure, but also soil pressure loads as they act to retain the approach roadway embankments (note that soil pressures can increase during seismic events).

There are several different basic types of abutments, which can be broadly categorized for discussion purposes as conventional, semi-integral, and integral abutments. Each type will be described in some detail below.

The choice of which of these three abutment types to use is influenced by several parameters, including bridge geometry (e.g., bridge length, skew, etc.), other geometric constraints (e.g., required horizontal clearances, fill slopes in front of the abutment, required vertical clearances, etc.), anticipated loads, future maintenance concerns, and local owner-agency preferences.

7.1   Conventional Abutments

Conventional abutments, sometimes called seat-type abutments, are characterized by these features: a joint separating the bridge deck from the abutment backwall and approach slab/approach pavement, and separation of the superstructure from the abutment by a bearing device of some kind.

These separations simplify the design of conventional abutments and can simplify their construction as well since the superstructure and the substructure are treated independently with a well-defined, easily-constructed interface. On the other hand, conventional abutments require the use of expansion joints, which require maintenance and have the potential to jam or to leak, leading to greater potential for deterioration of the girders, bearings, or abutments. In addition, some redundancy and robustness found in integral or semi-integral abutments is not found in conventional abutments.

7.1.1   Design Considerations for Conventional Abutments

There are numerous issues to consider in the layout and design of conventional abutments. Several are listed below:

Height

Conventional abutments can be broadly categorized in terms of height as either stub abutments or tall abutments (a.k.a., "deep abutments"). In a stub abutment, the depth of the abutment cap is set at a nominal, and relatively shallow, dimension, typically not much deeper than the cap width. The designs for stub abutment caps depths are often standardized based on owner-agency preference. Stub abutments usually also feature a header slope (a.k.a., a "spill-through" slope) in front of the abutment cap. The grade of the header slope can be as shallow as 4:1 or shallower or as steep as 1:1 or steeper, depending on owner-agency preferences, aesthetic considerations,

clearance considerations, and, slope stability considerations (including consideration of any riprap or slope protection that may be provided).

Tall abutments, on the other hand, function as retaining walls as well as supports. Tall abutments are often used when horizontal clearance requirements below the bridge prohibit the use of a header slope, or where limits to the superstructure span lengths restrict the location of the abutment. Figure 10a and 10b summarizes theses conventional abutment types.
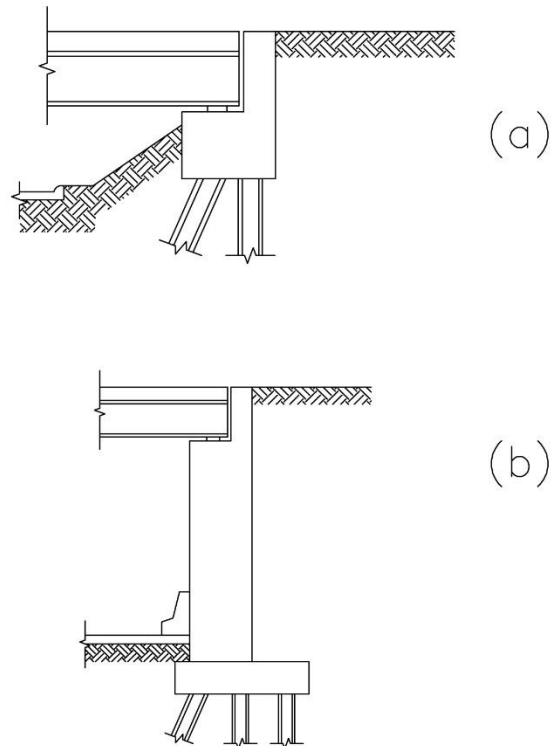
Figure 10 Conventional abutments of various heights: a) stub abutment; b) tall abutment.

Sometimes, a stub abutment is used in conjunction with a separate retaining wall in front of the abutment. This can be done to achieve similar geometric goals as a tall abutment in terms of maintaining horizontal clearances. Any of several retaining wall types can be used: Mechanically Stabilized Earth (MSE) walls, soil nail walls, drilled shaft walls, etc.

There are a few caveats, however. First of all, careful coordination is required to verify the abutment foundation elements do not interfere with any of the wall elements, especially when the wall uses straps, tie-backs, soil nails, etc. Also, the wall designer needs to be fully informed of the abutment configuration when designing the wall to make sure that loads are correctly quantified in the wall analysis. At a minimum the abutment and its backfill represent a surcharge loading on the wall; depending on the abutment and the wall configurations, additional loads may exist as well.

Width

The width of an abutment cap is controlled by several considerations, including:

- The need to accommodate bearings and anchor bolts with adequate edge distances, located to allow enough clearance between the girder ends and the backwall.

- The need to fit one or more rows of piles or drilled shafts with adequate spacing and edge distance.

- The need to meet seismic detailing guidelines related to required seat widths.
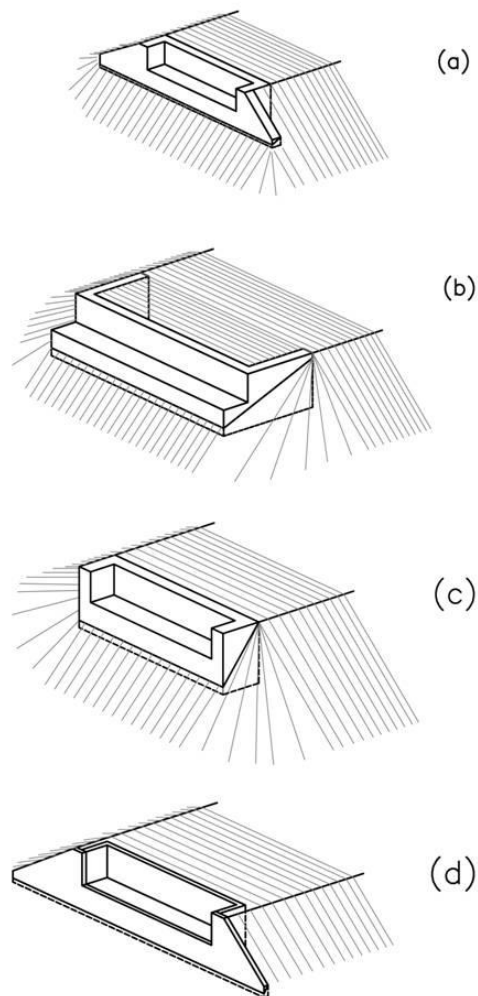
Figure 11 Abutment wingwall configurations.

Wingwall Configuration

Wingwalls are provided to retain the backfill which would otherwise "spill around" the ends of the abutment backwall and cap. Wingwalls can be oriented in a number of directions including parallel to the cap, angled at some angle (e.g., 30 deg., 45 deg.), turned back (parallel to the roadway, pointed away from the span), or turned forward (parallel to the roadway, pointed toward the span). The preferred orientation and layout of wingwalls is usually determined by owner-agency preference or local practice. For clarity, Figure 11a to 11d above depicts a variety of different wingwall configurations. Figure 11a) depicts stub abutment with wingwalls parallel to the abutment cap; Figure 11b) depicts stub abutment with turned back wingwalls; Figure 11c) depicts stub abutment with turned forward wingwalls (a.k.a. "ear walls" or "cheek walls"); Figure 11d) depicts stub abutment with MSE retaining wall and wingwalls parallel to the abutment cap.

Expansion Joints

Expansion joints should be designed for the anticipated movements of the superstructure relative to the abutment. Care should be taken to account for various potential sources of movement. Thermal expansion and contraction are typically the primary contributors to movements at expansion joints, but other sources may exist, particularly for longer structures or structures subjected to seismic events.

Expansion joints also need to be designed structurally for anticipated vehicle loading, although in many cases this boils down to selecting an appropriate joint from a selection of standard owner-agency or vendor designs based simply on the anticipated traffic.

Many different joint types exist, satisfying a wide range of design criteria. Since joints require maintenance, it is generally advisable to have significant input from the owner-agency regarding selection of the appropriate joint type; many owner-agencies have very explicit policies and guidelines on this topic.

Backfill

The selection of appropriate backfill for abutments has been the topic of considerable discussion. Typically, the backfill requirements for abutments vary significantly from state-to-state, and sometimes even from county-to-county or city-to-city. Most owner-agencies have standardized details and specifications regarding backfill for abutments. Some require cement-stabilized backfills, others require free-draining granular backfills, sometimes reinforced with geotextile fabric, and so on. Backfill requirements often also include provisions to facilitate drainage, as will be discussed further below. Designers are advised to consult with the appropriate owner-agency and/or the project's geotechnical engineer regarding backfill provisions for abutments.

Approach Slabs

The use, design, and detailing of approach slabs varies significantly among different owner-agencies and designers, different regions of the country, and different structure configurations (such as integral vs. non-integral abutments, etc.). Approach slab lengths, design methods, detailing preferences, how to deal with skews, how to connect to the abutment backwall, and even whether or when to provide approach slabs – are issues for which the answers vary significantly from one owner-agency to the next. Designers are advised to consult with the appropriate owner-agency regarding approach slab guidelines.

Drainage

Drainage issues can be broken into two categories: detailing for drainage of the top surfaces of the abutment and detailing for drainage of the backfill behind the abutment. For drainage of the top of the abutment, simple common-sense rules should be followed to provide positive-draining slopes so that water cannot collect or pond on the top surface of the abutment.

For drainage of the backfill behind the abutment, owner-agencies often have standard drainage details included either with their preferred abutment details or with their preferred backfill details. These may include the use of free-draining granular backfill materials, drainage strip materials placed against the abutment backwall, weep holes in the abutment, underdrain systems, etc.

7.1.2   Design Elements in Conventional Abutments

The following elements of conventional abutments require some degree of design:

Abutment Caps

Abutment caps (also sometimes called abutment stems) should be designed for vertical loading as beams spanning between foundation elements, including consideration of vertical moment and shear via a traditional section model design approach, or via a strut-and-tie method design approach, depending on the abutment cap's span/depth ratio and on owner-agency design policies. Note that design of the abutment cap as a beam may be unnecessary for abutments founded on spread footings.

The design may need to address torsion as well, particularly if the abutment or its backwall are particularly tall, or if there is significant eccentricity between the centerline of application of superstructure reactions (centerline of bearing) and the centerline of the foundations.

Abutment Backwalls

Abutment backwalls are the portion of the abutment above the beam seat (i.e. above the abutment cap or stem). Backwalls are typically designed as cantilever retaining walls carrying

lateral soil pressure from the backfill. Some agencies also require application of braking or tractive forces from the approach slab to the top of the backwall.

Abutment Wingwalls

Most designers exercise some discretion in the design of wingwalls due to the complex nature of their support and connection to the rest of the abutment. Designers are encouraged to develop rational design procedures based on careful and realistic consideration of the particular detailing of the wall. Some wingwalls, particularly longer walls, rest on pile or drilled shaft foundations; some use spread footing foundations; some are cantilevered off the abutment cap. Some are square and some are tapered, such as is shown in Figure 12 below of an MSE retaining wall. Most wingwalls will generally behave in a manner that can be adequately captured by a conservative, simplified cantilever beam analyses. In other cases, an analysis based on plate theory may be more appropriate.

In many cases, the design of abutment wingwalls may be standardized and presented in the owner-agency's design policy manual or design standards.

Figure 12 Conventional stub abutment with a parallel MSE retaining wall.

Abutment Foundations

The analysis of the abutment cap should include consideration of the foundation system and should include calculation of the foundation loads for separate use in the foundation design. Different designers and different owner-agencies have adopted various approaches to the calculation of abutment foundation loads. In some cases, the abutment cap is assumed to act as a rigid body, equally distributing vertical loads among the piles. In other cases, the abutment cap is assumed to act as a continuous beam on pin supports, with the distribution of load to each pile calculated based on standard beam theory. Consult owner-agency policy or discuss the anticipated behavior of the foundations with the project's geotechnical engineer to determine the most appropriate approach for calculating the distribution of loads in the abutment foundation system.

7.1.3   Forces on Conventional Abutments

Conventional abutment design should consider the following forces:

- Abutment cap self weight

- Abutment backwall self weight

- Abutment wingwall self weight

- Miscellaneous dead loads (bearing seats, lateral restraints, etc.)

- Superstructure dead load (including girders, cross-frames, deck, barrier rails, medians, overlays, provisions for future overlays, etc.)

- Approach slab dead load

- Superstructure live load

- Approach slab live load

- Lateral soil pressure (including consideration of groundwater as appropriate) on the backwall

- Lateral soil pressure (including consideration of groundwater as appropriate) on the wingwalls

- Live load surcharge

- Longitudinal forces (in select cases, depending on the nature of the bearings or integral connection provided between the superstructure and the abutment).

- Seismic loads

7.1.4   Detailing Considerations for Conventional Abutments

Abutments can be complicated to detail. There are many different elements converging in one region in an abutment, requiring care and thoroughness in their layout and detailing.

Approach Slab Connections

Different owner agencies have different details for the connection of approach slabs to abutments. Some prefer full moment connections with a double row of reinforcing. Some prefer a single row of reinforcing and detail the connection as a hinge, sometimes with other provisions such as the use of bond-breaker materials. Some prefer not to provide a positive connection

between the abutment and the approach slab. Designers are advised to review their owner-agencies preferences for detailing of this connection and to try to keep their design assumptions consistent with the detailing used.

Expansion Joints

There are so many variations on expansion joints that it is difficult to write a few rules of thumb for detailing abutments at expansion joints. The designer is simply advised to study the provided expansion joint details carefully and to detail the abutment appropriately to adequately accommodate and anchor the expansion joint. Note that many states recommend the use of blockouts and a second concrete, grout, or elastomeric concrete placement for the expansion joints. Sometimes anchoring reinforcing is provided within this blockout, sometimes it is not.

Construction Joints

Construction joints should be judiciously provided in abutments. Usually a construction joint is provided between the abutment cap and the abutment backwall. Sometimes construction joints are provided at the wingwall interface. Construction joints are often also provided in very long (wide) abutments to facilitate placement of concrete in more manageable volumes.

Bearing Seats

Various owner-agencies have individual preferences for the detailing of bearing seats. Some like to step the entire width of the abutment cap and to run the seat to the next girder. Other states prefer individual, isolated bearing seats (pedestal seats) of much smaller size, often only large enough to accommodate the bearings and perhaps the anchor bolts.

Bearings

Bearing design itself is a complex topic – many good references exist. For the purposes of this discussion it is enough to advise designers to make sure that their bearing seats are detailed to accommodate the proposed bearings. This may require specific sizing of the bearing seats, provisions for inset of the bearings into the bearing seats, adequate seat length for anticipated seismic displacements, future jacking, etc. Bearing design is discussed in more detail in NSBA's *Steel Bridge Design Handbook: Bearing Design* [21].

Anchor Bolts

Anchor bolts should be sized to resist horizontal shear loads (arising from sources such as braking forces, wind loads, seismic loads, etc.), and vertical pullout loads (as applicable). In addition to designing the anchor bolt itself to resist these loads, the embedment of the anchor bolt into the concrete should be evaluated. The AASHTO LRFD BDS [1] addresses the design of anchors in concrete, providing some direct requirements and also deferring to ACI 318 [22] for some requirements. In many cases, owner-agencies have standard details or standard design/detailing requirements for anchor bolts.

The detailing of anchor bolts, masonry and sole plates, and bent cap reinforcing should be carefully coordinated to minimize risk of conflicts during construction. In addition, mis-location and misalignment of anchor bolts can result in problems that are costly and difficult to resolve, so the use of details that facilitate anchor bolt adjustment during construction is encouraged if permitted by the owner-agency policy. Some suggestions include:

- Providing anchor bolt blockouts in the abutment cap (a.k.a. "grout cans") – A blockout (typically a round tube with a closed bottom) with dimensions significantly larger than that of the anchor bolt is provided in the abutment cap. Later, when the superstructure has been erected, the anchor bolt location is adjusted as needed and the blockout is filled with grout to lock the anchor bolt in the correct position.

- Using field welded bearing connections – The sole plate is detailed to be attached to the girder bottom flange using a field welded connection, allowing the position of the bearing, relative to both the girder and the anchor bolts, to be adjusted in the field.

Reinforcing

Reinforcing in abutments should be detailed following generally accepted detailing practices. Special care should be paid to detailing the connection of the wingwalls to the cap and backwall to avoid reinforcing conflicts and congestion and still provide a sound connection.

Pile Embedment

When piles are used as the foundation for an abutment, they are typically embedded at least 9" or more into the cap. In many cases, spiral or hoop reinforcing is provided around the embedded pile as confinement reinforcing, and in some cases a nominal mat of reinforcing is provided directly above the pile. These detailing practices are usually based on owner-agency preferences.

Drilled Shaft Reinforcing Embedment

As mentioned in the drilled shaft discussions in Section 5.3 of this chapter, the vertical reinforcing projecting from a drilled shaft into the abutment cap is usually detailed to be fully developed in the cap.

Battered Piles

Depending on the abutment height and the resulting lateral soil pressure on the abutment, it may be necessary to provide battered piles (sometimes called "brace" piles) to resist the applied horizontal forces. Often, the number of battered piles is selected so that the sum of the horizontal components of the pile axial load (associated with permanent loading effects) in the battered piles balances the net horizontal force due to active soil pressure on the abutment backwall and cap. When multiple rows of piles are used, usually only the front row includes battered piles; this takes advantage of the higher permanent vertical load on the front row piles due to overturning

effects (thus generating higher axial loads in the piles and therefore more lateral resistance when battered), and also avoids geometric conflicts between the battered and vertical piles. In more complicated cases, where the horizontal loads are more significant than can be easily resisted by the horizontal component of load in the battered piles, consideration of soil-structure interaction (addressing the lateral resistance of the pile in the soil) can be considered.

Wingwall Piles/Drilled Shafts

Depending on analysis, engineering judgment, or owner-agency policy, wingwalls over a certain length may require their own foundation by means of piles or drilled shafts. The intent of such foundations is to provide both additional vertical support to these long walls and to provide some measure of horizontal support along the base of the wall. These foundation elements are sometimes located directly below the wingwall and in other cases are located in a concrete protrusion (sometimes called a "blister") on the side of the wingwall. Usually the preferred scheme is based on owner-agency preference or local practice.

7.2   Integral Abutments

Integral abutments are a class of abutments in which the superstructure is integrally connected to the abutment and the abutment foundation. Generally, the girders are set on a previously cast abutment cap and then a closure pour is cast which encases the ends of the girders such that the girder ends are embedded into the abutment concrete, creating a "jointless" substructure. In some cases, there are other positive connections provided, such as reinforcing running through holes in the girder webs or anchor studs welded to the girders and embedded in the abutment concrete.

Integral abutments are different from semi-integral abutments (discussed in Section 7.3 of this chapter) because, as opposed to semi-integral abutments, there is no intentional moment relief detail (hinge) anywhere between the superstructure and the abutment foundation. However, much of the guidance provided for integral abutments is applicable for semi-integral abutments as well.

Integral abutments are most typically founded on a single line of vertical steel H-piles, although integral details have occasionally been used with piles, drilled shafts and spread footings. Steel H-piles can often be shown to provide both acceptable vertical load capacity and enough horizontal flexibility to accommodate longitudinal bridge movements without developing large restraining forces. In some cases, the piles are installed in predrilled, permanently cased holes filled with loose sand to achieve adequate pile flexibility if the natural soils are too stiff to allow sufficient pile flexibility. Horizontally stiff foundation systems which are inherently designed to restrict horizontal movement and/or abutment rotation (such as battered pile foundations or multiple rows of piles) are not a good choice for integral abutments.

The variations in terms of configuration and design methodology for integral abutments are myriad and affect mostly the design of the abutment and foundation, not the superstructure per se. The reader is directed to a compendium document, the *Proceedings of the 2005 FHWA Conference on Integral Abutments and Jointless Bridges* [23], which contains numerous papers

on integral and semi-integral abutments. Numerous other papers, reports, and guidelines address integral abutment design. Many owner-agencies include specific design guidance in their design policy manuals. An older, but simple and clear, treatment of integral abutment design can be found in "Integral Abutments for Steel Bridges,' in Volume II, Chapter 5 of the *AISI/NSBA Highway Structures Design Handbook* [18].

7.2.1 Design Considerations for Integral Abutments

There are numerous issues to consider in the layout and design of integral and semi-integral abutments. Several are listed below:

Height

Integral and semi-integral abutments can be broadly categorized in terms of height as either stub abutments or tall abutments. Most of the considerations mentioned previously in this chapter for conventional abutments apply to integral and semi-integral abutments as well. Some owners limit the beam depth that can be used for an integral abutment.

In addition, when integral or semi-integral abutments are located behind a retaining wall (such as in the case shown in Figure 12) the wall designer needs to be fully informed of the abutment configuration when designing the wall, in order to make sure that loads are correctly quantified in the wall analysis. Great care should be taken in designing integral abutments in conjunction with retaining walls to either: a) carefully and rigorously calculate the loading effects on the wall, particularly loading effects caused by integral abutment movements; or b) isolate the piles from the surrounding soil above the bottom of the retaining wall, sometimes accomplished by surrounding the piles with a compressible fill material retained by oversize pipe sleeves around the pile.

Width

The considerations mentioned previously in this chapter for conventional abutments apply to integral abutments as well.

Wingwall Configuration

Many of the considerations mentioned previously in this chapter for conventional abutments apply to integral and semi-integral abutments as well. Note that for integral and semi-integral bridges, wingwalls are typically shorter, do not use piles or drilled shafts for support, and are typically tapered rather than square. Some designers and some owner-agencies have advocated providing an expansion joint between the wingwalls and the abutment cap and backwall, in order to allow for more freedom of rotation in the abutment cap. However, others have reported no problems with fully connected, turned back, cantilever wingwalls with lengths up to 20' and heights up to 10' [18].

Expansion Joints

Expansion joints for integral and semi-integral bridges are typically provided at the interface between the approach slab and the approach roadway pavement, usually with a sleeper slab to support the far end of the approach slab and approach roadway pavement. Usually only a nominal expansion joint is provided at the interface between the approach slab and the pavement.

Backfill

Backfill requirements for integral and semi-integral abutments should be evaluated on a case-by-case basis. Many options exist, including options that reduce the unit weight or other properties of the backfill in order reduce active and passive pressures on the abutment.

Approach Slabs

Integral and semi-integral abutment bridges should use an approach slab. The approach slab should be detailed with some type of positive connection to the abutment by means of mild reinforcing. This is recommended to avoid problems associated with opening of the joint between the approach slab and the backwall. Many designers prefer to use a single row of reinforcing bars for this connection to create a hinge detail which allows the abutment to rotate more freely without inducing undesirable stresses in the approach slab. Other approach slab design and detailing recommendations mentioned previously in this chapter for conventional abutments apply for integral and semi-integral abutments as well.

Drainage

The considerations mentioned above for conventional abutments apply to integral and semi-integral abutments as well.

7.2.2   Design Elements in Conventional Abutments

The following elements of integral and semi-integral abutments require some degree of design:

Abutment Caps

Caps should be designed for vertical loading as beams spanning between foundation elements, including consideration of vertical moment and shear via a traditional section model design approach, or via a strut-and-tie method design approach, depending on the abutment cap's span/depth ratio and on owner-agency design policies.

In addition, as described below for abutment backwalls, the abutment cap is typically modeled as a horizontal beam between the girders, carrying lateral soil pressure from the backfill.

The design may need to address torsion as well, particularly if the abutment or its backwall are particularly tall, or if there is significant eccentricity between the centerline of application of superstructure reactions (centerline of bearing) and the centerline of the foundations.

Abutment Backwalls

Unlike backwall design for conventional abutments, backwall design for integral and semi-integral abutments typically models the backwall as a horizontal beam between the girders, carrying lateral soil pressure from the backfill, if the backfill is placed after the deck closure pour is placed around the girders. Alternately, if a partial thickness backwall and the backfill are placed prior to setting the girders and placing the closure pour, that partial thickness backwall is typically designed first as a cantilever retaining wall carrying lateral soil pressure from the backfill, and then is also evaluated as a horizontal beam carrying lateral soil pressure. The evaluation of the lateral soil pressure in an integral abutment should consider the effects of the development of some level of passive soil pressure as the abutment is pushed into the backfill by thermal expansion of the superstructure under conditions of higher temperatures.

Some owner-agency design policies require providing a layer of compressible material (expanded polystyrene or similar) of sufficient thickness to accommodate much of the movement in an integral abutment without generating large passive resistance pressures.

Abutment Wingwalls

Most designers exercise some discretion in the design of wingwalls due to the complex nature of their support and connection to the rest of the abutment. Designers are encouraged to develop rational design procedures based on careful and realistic consideration of the particular detailing of the wall. Some wingwalls, particularly longer walls, include pile or drilled shaft foundations, some use spread footing foundations, and some are cantilevered off the abutment cap. Some are square and some are tapered. Some will behave in a manner adequately captured by some conservative, simplified strip-beam analyses. In other cases, an analysis based on plate theory may be more appropriate.In many cases, the design of abutment wingwalls may be standardized and presented in the owner-agency's design policy manual or design standards.

When wingwalls are provided for integral abutments, care should be taken that the wingwalls do not inadvertently act to "anchor" the movement of the integral abutment. For instance, in cases when wingwalls are too long to be cantilevered off the abutment cap and piles or other foundation elements are needed to support the wingwall vertically, the wingwall should be separated from the abutment cap by means of an expansion joint so that the wingwalls do not restrict movement of the integral abutment. In such cases a full, independent foundation system may be needed for the wingwalls.

Abutment Foundations

The analysis of the abutment cap should include consideration of the foundation system and include calculation of the foundation loads for separate use in the foundation design.

Different designers and different owner-agencies have adopted various approaches to the calculation of abutment foundation vertical loads. In some cases, the abutment cap is assumed to act as a rigid body, equally distributing vertical loads among the piles. In other cases, the abutment cap is assumed to act as a continuous beam on pin supports, with the distribution of load to each pile calculated based on standard beam theory. Consult owner-agency policy or discuss the anticipated behavior of the foundations with the project's geotechnical engineer to determine the most appropriate approach for calculating the distribution of loads in the abutment foundation system.

Perhaps more important in integral and semi-integral abutment design is the calculation of horizontal forces and moments in the foundation elements. Appropriate design methodologies have been the subject of some debate; designers are encouraged to have open discussions of this issue with their appropriate owner-agency prior to beginning a design.

The range of design methodologies is large. Some have reported success by simply calculating vertical pile loads, ignoring any horizontal force effects, and keeping the bridge length and other geometry parameters within specified limiting values.

However, a more comprehensive analysis is usually warranted and would consist of careful calculation of superstructure movements and other horizontal force effects on the abutments, combined with a nonlinear soil-structure interaction analysis of the foundation elements. A simplified way to approach this is to separate the foundation analysis from the rest of the structure and consider the foundation elements independently. For the case of pile or drilled shaft foundations, this lateral analysis would be accomplished via a laterally loaded pile analysis, often facilitated by a standardized computer model based on p-y curve analysis of the lateral response of the pile in the soil which can be accomplished using commercial soil-structure interaction design software or a soil-structure interaction analysis performed in a finite element analysis model with nonlinear soil-springs. This is typically run as a displacement-driving loading condition, where the forced displacement is based on the thermal expansion of the superstructure when the temperature rises above the assumed temperature at time of construction.

A more rigorous approach to a comprehensive analysis might involve the modeling of the soil response directly in the structural analysis model. This step eliminates the tedious iterations of exchanging information manually between the geotechnical and the structural analysis models, but the resulting soil-structure interaction model can become complex.

In either case, designers are encouraged to involve a qualified geotechnical engineer in the determination of the soil response to the various force- and displacement-driven loading conditions. Discussions should include the soil behavior and the structure behavior since the laterally loaded pile analysis results are heavily dependent on the boundary conditions assumed (e.g., "fixed pile head condition" vs. "free pile head condition" vs. other, more refined modeling of moment and translational stiffnesses).

Often a simple 2D model is adequate for the soil structure interaction analysis. If there is significant skew, a full 3D analysis may be warranted. However, many owner-agencies place

limitations on the severity of skew that will be permitted in integral abutment bridges, so this level of modeling complexity is probably not necessary in most cases.

Once the analysis method is selected, there are still many issues to be addressed. For example, in some integral abutments founded on steel piles, the calculated longitudinal movements of the bridge might result in bending moments in the pile (typically at the point of embedment into the abutment cap) that exceed the yield moment of the pile. In those cases, it is common to allow a plastic hinge to form (which provides significant moment relief for any movements above those which cause yielding of the piles) rather than resizing to a larger (and thus stiffer) pile.

Simultaneously, many designers choose to check the piles through their embedment depth in the soil to determine if they satisfy the requirements for bending-axial interaction outlined in the AASHTO LRFD BDS [1] for steel beam-columns, conservatively assuming the soil does not provide any bracing for the pile, and using the distance between moment inflection points as the unbraced length [18]. Some designers have pointed out that allowing a plastic hinge at the pile-abutment interface while simultaneously sizing the pile to prevent even a nominal overstress in terms of bending-axial interaction lower in the pile represents an inconsistent design approach, but to date there have been no known significant in-service problems for piles designed in this fashion. Full resolution of this debate is beyond the scope of the Steel Bridge Design Handbook. Designers are encouraged to consult the most current design guidelines and research papers for the latest positions on this and related issues [e.g., 23, **Error! Reference source not found.**].

While much of the above discussion focused on design issues for steel pile foundations for integral and semi-integral abutments, many of the same discussions apply to integral and semi-integral abutments with drilled shaft or spread footing foundations as well. However, these foundation types do not exhibit the same ductility as steel piles, and so allowing plastic behavior is not advised.

7.2.3 Superstructure Design Considerations for Integral Abutment Bridges

Traditionally, the general assumption has been to ignore any support restraint from the integral abutment when designing the superstructure. However, designers are warned that this may not be conservative in all cases. In the past, it has been considered universally conservative for superstructure design to consider the abutment support as a pinned support. While this is conservative for analysis of the positive moment region of the girders, it is not necessarily conservative at the ends of the girders. At the ends of the girders where they frame into the integral abutments, some degree of negative moment may develop, depending on the relative stiffness of the integral abutment and its foundation. Neglecting this potential for negative moment development may result in undesirable deck cracking or overstress of the girder flanges.

In the case of steel girder bridges with integral abutments, the consequences of these overstresses do not represent life safety issues. If there are overstresses at the ends of the girders, the inherent ductility of the steel girders will allow a hinge to form and the girder will behave as if it were simply supported at that end, as was originally assumed in the superstructure design. The main concerns associated with such overstresses will be related to serviceability, particularly with

regard to the potential for tension to occur in the concrete deck in the vicinity of an integral abutment. For this reason, many owner-agencies have design policies or standard details which include additional longitudinal reinforcing in the deck in the vicinity of integral abutments.

If there is reason for concern that simple detailing provisions may not be sufficient, a suggested redundant design procedure would be to initially design the superstructure assuming simple support conditions at integral abutments – this analysis should be used for the initial sizing of the girders. Then, a second analysis can be performed where the integral abutment and its foundation are included in the overall superstructure analysis to model the frame action at the integral abutments. The results of this second analysis can be used to check the girder design at the integral abutments and to size the deck reinforcing and its continuation into the abutment backwall / cap. However, this level of rigor is generally not warranted for most bridges.

7.2.4   Forces on Integral Abutments

Integral abutment design should consider the following forces:

- Abutment cap self weight

- Abutment backwall self weight

- Abutment wingwall self weight

- Miscellaneous dead loads (bearing seats, lateral restraints, etc.)

- Superstructure dead load

- Approach slab dead load

- Superstructure live load

- Approach slab live load

- Lateral soil pressure on the backwall (active & passive)

- Lateral soil pressure on the wingwalls

- Longitudinal applied forces (in select cases, depending on the nature of the bearings provided between the superstructure and the abutment).

- Induced forces due to longitudinal movements (most importantly thermal movements)

- Seismic loads

Note that superstructure loads such as dead load and live load can potentially apply moments (or rotations) to the integral abutments, depending on the stiffness of the abutment and the abutment foundation. In the past, these effects have been ignored by many designers, based on the assumption that the superstructure is usually significantly stiffer than the integral abutment and its foundation. However, designers are warned that this is not always the case. Thus, some investigation of this issue, at least in a cursory manner, is advisable.

7.2.5   Detailing Considerations for Integral Abutments

In some ways, the detailing of the concrete for integral abutments can be much simpler than for conventional abutments, because the shapes of the various concrete elements are often much simpler. However, great care should be taken when detailing the interface between the abutment, wingwalls and approach slabs to permit adequate movement of the structure while retaining the backfill adequately. Typical detailing considerations are provided below and shown in Figure 13. A depiction of a completed integral abutment is provided in Figure 14 for reference as well.

Approach Slab Connections

Different owner agencies have different details for the connection of approach slabs to integral and semi-integral abutments. Most prefer to detail the connection as a hinge, with a single row of reinforcing and sometimes with other provisions such as the use of bond-breaker materials. Designers are advised to review their owner-agencies' preferences for detailing these connections and to try to keep their design assumptions consistent with the detailing used.

Expansion Joints

For integral and semi-integral abutments, expansion joints are typically provided only away from the abutment, at the interface between the approach slab and the roadway pavement. In many cases, at these locations only a nominally sized expansion joint is provided.

Construction Joints

Construction joints should be judiciously provided in integral and semi-integral abutments. Usually a construction joint is provided between the abutment cap and the abutment backwall. Sometimes construction joints or expansion joints are provided at the wingwall interface. Construction joints are often also provided in very long (wide) abutments to facilitate placement of concrete in more manageable volumes. Depending on the intended construction sequence, a vertical construction joint may be provided between the partial thickness backwall and the closure pour around the girder ends.

Bearing Seats

In integral and semi-integral abutments for steel girder bridges, bearing seats per se are not usually provided.

Bearings

For steel girder bridges with integral abutments, traditional bearings typically are not provided since there is no relative movement or rotation between the girders and the abutment cap. Instead, typically a nominal leveling pad or unreinforced neoprene pad is provided, along with anchor bolts. Alternately, some designers and owner-agencies prefer using "heavy" bolts and nuts which function as vertical supports as well as anchor bolts. The leveling pad or bolts should be designed as columns to resist the self-weight of load of the girders, the deck, and the closure pour. The leveling pad detail is simpler to construct, but the bolted details offers the advantage of facilitating vertical adjustment of the ends of the girders via adjustment of the support nuts.

Anchor Bolts

Many traditional design rules and suggestions for anchor bolts do not apply directly in integral and semi-integral abutments. However, many traditional rules related to detailing and providing adjustability for anchor bolts and their connections to the rest of the structure can be adapted for use in integral and semi-integral abutment bridges. See the suggestions listed above for Anchor Bolts in conventional abutment bridges.

Figure 13 Example of integral abutment details, prior to casting the closure pour.
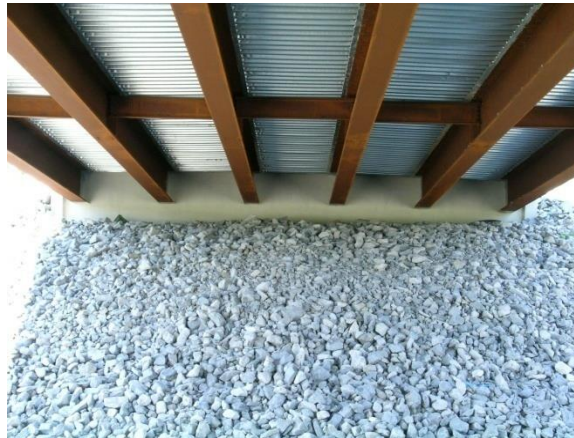
Figure 14 Photograph of a completed integral abutment.

Reinforcing

Reinforcing in integral and semi-integral abutments should generally be detailed following typical detailing practices. Some owner-agencies specify that reinforcing bars run continuously through holes in the girder webs. Special care should be exercised in detailing the connection of the wingwalls to the cap and backwall to avoid reinforcing conflicts and congestion, but to still provide a sound connection; note also that some owner-agencies might specify isolating the wingwalls from the abutment cap and closure pour to allow free movement of the cap and closure pour without subjecting the wingwalls to those same movements. Care should also be taken in detailing the reinforcing connecting integral abutments to the deck; some negative moment may develop in the deck in the vicinity of the integral abutment, as mentioned in Section 7.2.3 of this chapter.

Pile Embedment

When piles are used as the foundation for an abutment, they are typically embedded a specified distance into the cap. The connection of the piles to an abutment cap in an integral end bent bridge is a moment connection and sufficient embedment should be provided to develop that moment. Reference [18] includes an example of how to check the embedment depth. In most cases, spiral or hoop reinforcing is provided around the embedded pile as confinement reinforcing, and in some cases a nominal mat of reinforcing is provided directly above the pile. These detailing practices are usually based on owner-agency preferences.

Drilled Shaft Reinforcing Embedment

As mentioned in the drilled shaft discussions earlier in this chapter, the vertical reinforcing projecting from a drilled shaft into the abutment cap is usually detailed to be fully developed in the cap.

Battered (Brace) Piles

Battered piles are not typically used in integral abutments; however, they may be used in semi-integral abutments.

Wingwall Piles/Drilled Shafts

As mentioned previously, most designers and owner-agencies discourage or disallow the use of founded wingwalls.

7.3 Semi-Integral Abutments

Semi-integral abutments are a class of abutments in which the superstructure is integrally connected to the abutment backwall, but the abutment backwall is isolated from the abutment cap by means of some sort of hinge detail. Semi-integral abutments offer some of the advantages of fully integral abutments such as elimination of expansion joints and a robust end diaphragm detail for the superstructure, while also reducing the moment demand on the piles by providing a bearing or a hinge detail that allows the piles to behave in a free-head rather than a fixed-head manner (i.e., the top of the pile is free to rotate as well as to translate).

Much of the guidance provided above for integral abutments is applicable for semi-integral abutments as well. Some specific guidance related to semi-integral abutments is provided here, but a full discussion of semi-integral abutments is beyond the scope of this document. The variations in terms of configuration are myriad and affect mostly the design of the abutment itself. The reader is directed to a recent compendium document, the *Proceedings of the 2005 FHWA Conference on Integral Abutments and Jointless Bridges* [23], which contains numerous papers on integral and semi-integral abutments.

The superstructures for semi-integral bridges are sometimes supported on bearings as with a conventional structure, thus allowing longitudinal translation. In this case the backwall is separate from the abutment cap, yet the beam ends are encased in the backwall as in an integral abutment bridge. Details are developed to keep the structure backfill from working its way out between the backwall and the abutment stem. Bridges with this type of bearing-supported semi-integral abutments can be used for much longer bridges than fully integral abutments because the movement capacity is not limited by the pile movement/bending capacity.

In other cases, semi-integral abutments are designed with a concrete hinge detail that allows for girder end rotation but restricts longitudinal translation of the superstructure relative to the abutment cap. A commonly used detail features a concrete shear key to restrict translation with elastomeric pads to allow rotation.

An option for older bridges with conventional abutments it to retrofit them with semi-integral abutments in order to eliminate the deck joints above the beam ends while retaining most of the existing abutment structure.

7.3.1   Forces on Semi-Integral Abutments

See the discussion of Forces on Integral Abutments.

7.3.2   Detailing Considerations for Semi-Integral Abutments

See the discussion of Detailing Considerations for Integral Abutments.

8.0   PIERS (BENTS)

Interior supports for bridges (away from abutments/end bents) go by the title "pier" or "bent." There is some debate among engineers as to the exact meaning and usage of these terms, and often their definitions depend on local custom and/or owner-agency preference. For the sake of consistency within this chapter, the term *pier* will be used throughout, and is meant to refer to any structure which supports the superstructure at intermediate points between end supports.

8.1   Pier Types

There is a nearly limitless range of pier types. In some ways, it is easier to categorize pier types in terms of various combinations of a few basic pier elements, rather than trying to enumerate all the possible combinations individually. The basic pier elements can be classified as follows:

- Pier Caps – A more or less horizontal member, on which the superstructure rests. In some cases (such as wall piers), the pier cap and the pier vertical support(s) are one and the same. Pier caps can have a square cross section, a rectangular cross section, an "inverted T" cross section, or any of a variety of other shapes. Pier caps can be prismatic, tapered, flared, or stepped. Traditionally, most concrete pier caps to date have been cast-in-place, but precast pier caps have been successfully used and are gaining wider acceptance throughout the US. See Figure 15a to 15c for depictions of pier cap types for multi-column piers. Figure 15a) depicts a prismatic pier cap; Figure 15b) depicts a tapered pier cap; Figure 15c) depicts a pier cap with parabolic haunches. See Figure 16a to 16c for depictions of single column piers. Figure 16a) depicts a prismatic pier cap; Figure 16b) depicts a tapered pier cap; Figure 16c) depicts a pier cap with parabolic haunches
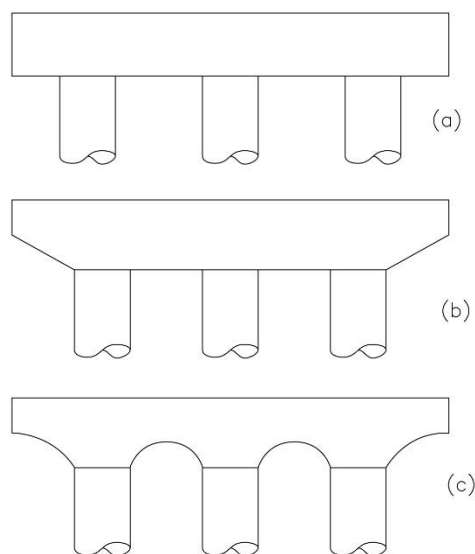
Figure 15 Various types of pier caps for multi-column piers.
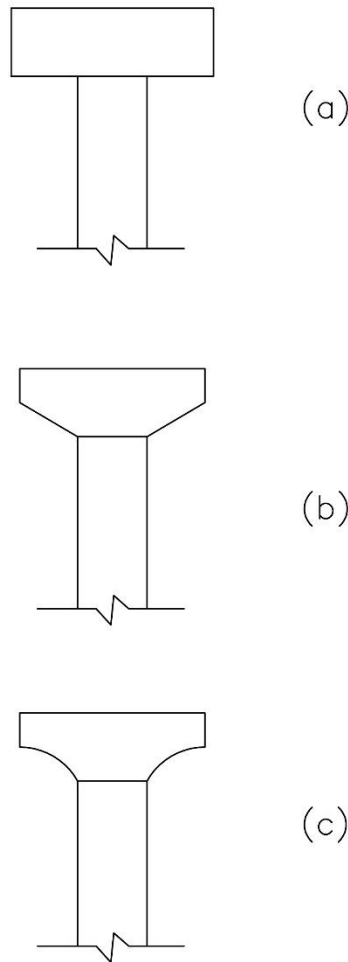
Figure 16 Various types of pier caps for single column piers.

- Pier Vertical Supports – Any more or less vertical member which supports the pier cap, and which rests on the pier foundation. In most cases, the pier vertical supports take the form of one or more columns, where columns are vertical members with cross sectional dimensions significantly smaller than the horizontal dimensions of the pier cap. In some cases, the pier vertical support takes the form of a single "wall", i.e., a member with cross sectional dimensions nearly the same as the horizontal dimensions of the pier cap. Pier vertical supports (whether walls or columns) can take any of a number of cross-sectional shapes, including round columns, square columns, rectangular columns, etc. Columns may be prismatic, tapered, stepped, or flared, and may be solid or hollow. Most concrete columns to date have been cast-in-place, but precast columns have been successfully used and are gaining wider acceptance throughout the US.

- Pier Intermediate Struts – Any more or less horizontal members which serve to brace the pier vertical supports (columns) to increase the column buckling capacity.

- Pier Intermediate Bracing – Any type of bracing, such as X-bracing, web walls (i.e., concrete shear walls between columns), etc., which serves the purposes of both: a) bracing the columns to increase buckling capacity; and b) providing a more efficient shear load path for carrying horizontal forces through the pier.

Each of the above elements can be fabricated using either steel or concrete (most commonly) or timber or masonry (much less common). In the case of steel, the elements may take the form of rolled sections, built-up open plate sections, built-up closed box sections trusses, lattice-work, or other configurations. In the case of concrete, the elements may be conventionally reinforced, prestressed or post-tensioned, or both, and may be either cast-in-place or precast.

Some of the more commonly used combinations of these various elements are listed below, but the list should not be considered comprehensive or limiting in any way:

- Reinforced Concrete Multi-Column Piers – Perhaps the most common type of pier, this type consists of a reinforced concrete cap supported by two or more reinforced concrete columns, depicted in Figure 17 and Figure 18 . Generally, the pier cap is only conventionally reinforced, but occasionally post-tensioning is used as well. Typically, the column spacing is determined to satisfy a balance between economical design of the pier cap and columns, although geometric constraints may control the arrangement in specific cases.

Figure 17 Typical pier with round columns and a pier cap stepped at each bearing.

Figure 18 Typical pier with round columns and a prismatic pier cap with individual pedestals for each bearing.

- Reinforced Concrete Single Column Piers – This type, sometimes called a "hammerhead pier" due to its resemblance to a hammer, consists of a reinforced concrete pier cap "hammer head" supported by a single reinforced concrete column. The use of post-tensioning in a hammerhead pier cap is more common than in a pier cap for a multi-column pier. This pier type is particularly popular for narrow bridges (where there is not room for two or more columns) and for tall piers (where a single, much larger column may provide a more efficient means to resist column buckling).

- Pile Bents – A pier cap supported on multiple steel or precast concrete piles is sometimes called a "pile bent." Typically, in a pile bent, there is no distinction between the "columns" and the "foundations" – the foundations are just continuations of the piles supporting the cap.

- Straddle Bents – A straddle bent is a multicolumn pier in which an extremely wide column spacing is used to allow for the passage of a roadway directly below the pier, such that the pier is "straddling" the roadway or other feature below the bridge. Due to the unusually wide column spacing and the resulting long span length of the pier cap, straddle bent pier caps are often specialty structures such as steel box beam caps or post-tensioned concrete caps.

- Integral Piers – It is occasionally desirable to construct pier caps integral with the superstructure. Sometimes this offers advantages in terms of structural efficiency, sometimes it offers aesthetic benefits, and sometimes it helps reduce structure depth and improve vertical clearances. Integral pier caps for steel girder bridges have been constructed both using steel [24] and using concrete [25].

- Steel Piers – While most piers are constructed from reinforced or prestressed/post-tensioned concrete, there are still situations calling for the use of steel for part or all of a pier. One common opportunity for using steel piers is for temporary bridge structures,

such as temporary access bridges on construction sites. In these cases, contractors often prefer using steel substructures since they are usually light weight and easy to handle, relatively quick to install, and potentially reusable. Other opportunities for using steel elements in piers include long span straddle bent caps and integral pier caps. Other applications of steel piers are perhaps less frequent, but can provide good solutions in the right context. For example, several steel girder bridges have been built with integral slant-leg steel piers or steel delta-piers to solve tough design problems in challenging sites. Another useful option is using concrete-filled steel tube piles, which offer many of the advantages of both systems when used in the right applications.

8.2   Selecting the Right Pier Type

Selection of pier type is usually heavily influenced by an assessment of the General Design Considerations listed below. However, other factors influence the selection of pier type, including:

Aesthetics

The wide range of pier types available makes piers an attractive candidate for aesthetic manipulation. In many cases, the type and shape of pier caps and columns ends up being dictated by aesthetic considerations such as corridor aesthetic themes or owner-agency or public preferences, such as is shown in Figure 19. Designers are encouraged to embrace rather than fight this trend. Often the cost of materials is not the driving factor in overall bridge project cost and in many cases aesthetic considerations can be included in a project at little or no additional cost. The key is for the structural engineer to actively participate in the aesthetic design process so that structural considerations are appropriately addressed early on.

Figure 19 Multi-column piers with haunched pier caps and curved columns with formliner treatments.

Local Contractor Preferences

Local contractor preferences are a key consideration in selecting pier types. In most cases, local contractors are limited in terms of equipment, materials, and experience/expertise to a relatively narrow range of substructure types which they can efficiently and economically build.

Local Site Conditions

Subsurface conditions affect foundation type selection which often has a direct impact on the pier type selection. Similarly, local climate conditions (proximity to saltwater, regular use of deicing salts, extreme temperature variations, etc.) can have a direct impact on selection of materials for piers.

Vehicle or Vessel Impact Considerations

The presence of roadways or railroads in the vicinity of a pier may require the designer to address vehicle impact loads in the pier design; in some cases, owner-agencies have preferences regarding pier type in situations where piers are exposed to higher probability of vehicle impact. The same considerations hold true for piers supporting bridges over navigable waterways, where vessel impact considerations may influence the selection of pier type.

8.3   General Design Considerations

There are many issues to consider in the design of piers. A few select considerations are listed below, but this list should not be viewed as all-inclusive. When designing any pier, designers are advised to carefully consider the various possible loading conditions, including displacement-driven as well as force-driven loading effects, structural connection details and how they influence the behavior of the pier, and the influence of foundation response on the behavior of the pier.

Height

The height of the columns supporting a pier obviously has a significant impact on their behavior and design. Height influences not only loads (taller piers generally have higher shear as well as higher moment demands; tall and slender columns, such as what is shown in Figure 20, can experience second-order moment magnification), but also capacity (buckling capacity is a function of the square of the column heights, moment capacity of some column cross sectional shapes is a function of unbraced length).

Column Proportions

A topic related to pier height is column proportions. Designers are cautioned to pay particular attention to column proportions; excessive slenderness can dramatically reduce axial capacity and can also lead to excessive second-order moment magnification effects (P-Δ effects).

The AASHTO LRFD BDS [1] (Article 5.6.4.3) allows that slenderness effects can be neglected in columns when KL/r is less than 22, provides an approximate method for evaluating moment magnification in columns when KL/r is between 22 and 100, and suggests that a second-order analysis (a P-Δ analysis) be used to evaluate moments in columns when KL/r exceeds 100.

Care should also be taken in determining the appropriate value of K, the effective length factor. Thorough understanding of the underlying assumptions in the development of K factors is key to correct analysis of slender columns. Elastic buckling capacity and second order effects are functions of the square of the effective length; changes to the K factor have significant impact on the design.

Figure 20 Single column hammerhead pier with tall columns over a deep valley.

Keep in mind also that columns should be evaluated about both axes, both in terms of loads and in terms of capacity. In multi-column bents, for example, the column axial capacity and the column moment behavior (shape of the moment diagram) will be significantly different in the transverse and longitudinal directions; in the transverse direction the columns and pier cap will interact as a frame, while in the longitudinal direction the columns will behave like flagpole cantilevers. In a skewed bridge it is often advisable to resolve the various longitudinal and transverse loading effects into components in the local pier orthogonal directions to more clearly associate the loads and the behavior of the pier.

Designers should also be aware that in many cases the total design height of a pier is greater than the clear height of the columns from pier cap to ground. In cases where a single drilled shaft is the foundation for each column, with no intermediate pile footing, particularly in cases where there are very soft soils with weak lateral resistance, the design height for the columns is the

distance from the pier cap (generally taken as from the pier cap neutral axis) to the "equivalent point of fixity" of the foundation (discussed previously in this chapter in the section on foundations).

Finally, aesthetics should be a consideration in proportioning columns. A good discussion of general bridge proportioning is provided by Gottemoeller [26].

Solid vs. Hollow Columns

Hollow columns offer some distinct advantages, but generally only in select, fairly rare situations. Hollow columns are most suited for use in very tall piers. When used in appropriate applications, hollow columns offer several advantages, including:

- Reduced dead load on foundations.

- Potential savings in construction efforts associated with lifting and placing heavy materials on tall piers in difficult terrain.

- Reduced stiffness (and resulting reduced internal loads from displacement-driven loading effects).

- Easier construction (if precast) – Hollow columns are often constructed using precast segments, post-tensioned together. This type of construction generally involves less field work and smaller equipment. However, if hollow columns are to be constructed using cast-in-place concrete, the need for inner and outer forms and the general difficulty of constructing thin concrete sections in the field, can make their use less desirable.

Poston, et al [27], Taylor, et al [28], and Corven [29] offer more discussion of hollow column design.

Column Spacing

Column spacing is ideally set to optimize the design of both the columns and the pier cap, or to satisfy basic aesthetic proportioning guidelines. However, in many projects other issues such as required horizontal clearances to lower roadways, constructability considerations and desire to duplicate details in long, multiple span bridges, need for foundations to clear subsurface conflicts such as existing utilities, etc., override these considerations. Many owner-agencies also have guidelines on preferred maximum column spacing. Overall, designers are advised to keep in mind the various implications associated with column spacing, since it has a significant impact on pier cap, column, and foundation design.

Column Analysis

As mentioned above, slender columns may require analysis of second-order moment magnification effects. The degree of rigor and refinement needed is generally a function of the

slenderness of the column. Article 5.6.4.3 of the AASHTO LRFD BDS [1] includes design provisions addressing the minimum required level of analysis based on the slenderness of the column. Moment magnification can be neglected for very stocky (nonslender) columns. For columns with $K\ell_u/r$ up to 100, the AASHTO LRFD BDS [1] provides a simplified method for estimating moment magnification in nonprestressed columns. For very slender columns with $K\ell_u/r$ greater than 100, a nonlinear analysis should be performed. This is typically accomplished using a second order geometrically non-linear analysis, more commonly called a P-Δ analysis. P-Δ analysis can be accomplished using a finite element analysis approach using any of a variety of commercial FEM programs. Or, for simpler cases (most routine bridges), P-Δ analysis can be performed using any iterative analysis approach, including via a simple spreadsheet. The analysis need only consider the incremental additional moments due to additional deflection from the previous iteration.

Pier Cap Proportions

Ideally, pier caps should be proportioned to efficiently and economically carry shear and moment effects. Other considerations such as aesthetic requirements or constructability issues may also influence the proportioning of pier caps.

Keep in mind also that there are minimum and maximum thresholds for span to depth ratios for pier caps, just as there are for any beam structures. For example, in extreme cases, excessively shallow pier caps may have deflection or constructability problems. More commonly, when pier caps end up quite deep; pier caps with low span-to-depth ratios warrant design as "deep beams" using strut-and-tie modeling (STM) techniques.

Much has been recently written about STM methods and many references exist. However, most older reports and guides are out of step with the current AASHTO LRFD BDS [1] provisions and their use is discouraged. Two more current research reports, which form the basis for the current AASHTO LRFD BDS [1] provisions, were published by the University of Texas at Austin in 2009 and 2012 [30, 31], and include clear, comprehensive explanations of STM design theory, derivation of the current AASHTO LRFD BDS [1] provisions, and design examples. Another excellent guide is the Reference Manual and design Examples for NHI Course 130126, Strut-and-Tie Modeling (STM) for Concrete Structures [32].

Tapering or haunching of pier caps is a common practice. There may be a perception that the savings in materials translates directly into a cost savings, but this is seldom true in modern times since the added complexity of tapered or haunched pier caps adds significant labor costs in terms of both shop effort (reinforcing bending) and field effort (cap forming, reinforcing assembly, quality control, etc.). Generally, the main advantages of tapering or haunching of pier caps is aesthetic, with a secondary benefit of reducing loads on pier cap shoring and formwork.

Bearing and Pier Fixity

A key consideration in layout of a bridge is determination of which piers will utilize fixed bearings versus expansion bearings. Bearing fixity dramatically affects the loading on piers.

Piers with fixed bearings are generally subject to much higher horizontal (particularly longitudinal) force effects than expansion piers.

However, the problem is not as simple as just saying "this pier is fixed and that pier is not." Even piers with expansion bearings are subject to some degree of horizontal force effects. For example, even if a pier is an "expansion pier" it generally still uses details which provide lateral restraint of the superstructure so that the pier will be subject to transverse load effects. Similarly, a fixed pier is not infinitely stiff due to the flexibility of columns and foundation elements. In the longitudinal direction, where differences in bearing fixity are most significant, it is important to understand that even expansion bearings will transfer some amount of longitudinal loading to the substructure. No bearing is completely frictionless or completely flexible. For example, "sliding" bearings (bearings featuring sole plates with low friction PTFE sliding surfaces) are still subject to some degree of friction, and piers with these types of bearings are generally designed for the lesser of the longitudinal force that would occur at that pier if the bearing where fixed or a static friction force based on the permanent vertical loads on the bearing multiplied by a conservative estimate of the coefficient of static friction. Similarly, "free" elastomeric bearings are not completely flexible; the distribution of longitudinal force to piers with these types of bearings should be based on an effective net stiffness considering the flexibility of the pier itself and the flexibility of the elastomeric bearing (derived from the bearing dimensions and the shear modulus of the elastomeric material).

In multispan bridges, the distribution of longitudinal forces to each pier should be based on a relative stiffness analysis (either a simplified analysis conducted using hand calculations, or a full bridge model, as appropriate). In longer, multiple span bridges, designers are encouraged to explore more than one arrangement of fixed vs. expansion piers. A balance can sometimes be found which optimizes the design of each pier, particularly when the pier heights vary significantly from pier to pier, by judiciously choosing which piers are fixed and which are expansion. The choice of which piers are fixed and expansion has a significant impact on the distribution of longitudinal forces in multi-span bridges. This can result in significant cost implications and should be studied early in design development.

8.4   Forces on Piers

Pier design should consider the following forces:

- Pier cap self weight

- Column self weight

- Miscellaneous dead loads (bearing seats, lateral restraints, etc.)

- Superstructure dead load (including girders, cross-frames, deck, barrier rails, medians, overlays, provisions for future overlays, etc.)

- Superstructure live load

- Centrifugal force from live load (on curved bridges only)

- Wind on superstructure

- Wind on substructure

- Wind on live load

- Wind overturning

- Longitudinal force from live load (a.k.a. longitudinal braking force)

- Differential earth pressure when constructed in slopes

- Shrinkage

- Thermal expansion/contraction

- Stream flow

- Ice loads

- Seismic loads

- Vessel/vehicle collision loads

For analysis of "routine" bridges, transverse forces on the superstructure (such as wind on superstructure, wind on live load, and centrifugal force, etc.) are assumed to be distributed among the piers and abutments based on a simple tributary span length distribution assumption.

On the other hand, longitudinal forces on the superstructure of a routine bridge (such as wind on superstructure, wind on live load, longitudinal braking force, etc.) are typically distributed among the piers and abutments based on either a simplified or a rigorous relative stiffness analysis. A simplified relative stiffness analysis can be performed by hand and simply models the flexural stiffness of each pier or abutment in the longitudinal direction. The flexibility of both the bearings and the columns over their entire height (down to the point of fixity) are considered. If appropriate, a more rigorous relative stiffness analysis can be performed using either a 2D or 3D computer model.

8.5   Multi-column Pier Considerations

Multi-column piers are perhaps the most common pier type in most routine bridge design. Their design is relatively straightforward and usually requires only a first-order analysis, although sometimes if columns are tall and/or slender, some consideration of second-order moment

magnification effects may be required. The analysis and design of multi-column piers can be accomplished using one of a variety of commercial pier design software programs, or can be performed using a simple frame model built using FEM analysis software, accompanied by hand or spreadsheet calculations and/or commercial software for the design of individual elements such as concrete columns.

Detailing of multi-column bents is fairly straightforward as well. Often, owner-agencies have guidelines or standard details they prefer. Local contractor preferences and past local practices should also be considered to achieve maximum economy. Some other specific suggestions include:

Construction Joints

Construction joints should be judiciously provided in multi-column piers. Generally, a construction joint is provided between columns and the pier cap and between the columns and the foundations. Construction joints are often also provided in very long (wide) pier caps to facilitate placement of concrete in more manageable volumes. If staged construction is required, care should be taken in detailing appropriate construction joints and provisions for connection of reinforcing between stages.

Bearing Seats

Various owner-agencies have individual preferences for the detailing of bearing seats. Some owners prefer to provide a full width flat step in the pier cap and to run the seat to the next girder. Other states prefer individual, isolated bearing seats (pedestal seats) of much smaller size, often only large enough to accommodate the bearings and perhaps the anchor bolts.

Bearings

Bearing design itself is a complex topic which deserves its own discussion – many good references exist (including other volumes in the Steel Bridge Design Handbook). For the purposes of this discussion it is enough to advise designers to make sure that their bearing seats are detailed to accommodate the proposed bearings and their anchorage as well as provisions for their replacement and seat width (relevant in seismic zones). This may require specific sizing of the bearing seats, provisions for inset of the bearings into the bearing seats, etc. NSBA's *Steel Bridge Design Handbook: Bearing Design* [21] presents a more detailed discussion of bearing design.

Anchor Bolts

The discussion of Anchor Bolts in Section 7.1.4 (Detailing Considerations for Conventional Abutments) of this chapter is equally applicable to multi-column piers.

Reinforcement, General

Reinforcement in piers should be detailed following generally accepted detailing practices, realizing that these practices are typically more stringent in seismic zones. Special care should be paid to detailing the projection of column vertical and confinement reinforcement to avoid reinforcement conflicts and congestion, but to still provide a sound connection. Simple techniques such as interrupting pier cap stirrup patterns at columns and discontinuing some of the pier cap positive moment reinforcement at columns can go a long way to relieving reinforcement conflicts and congestion.

Column Reinforcement Embedment into Pier Cap

The vertical reinforcement projecting from a column into the pier cap is usually detailed to be fully developed in the cap. This can be accomplished by means of straight reinforcement (full development length) or by fully developed hooks. Hooks are shorter (and may fit better in shallower pier caps) but they complicate the construction of the pier cap reinforcement cage.

Column Reinforcement Embedment into Spread or Pile Footings

The vertical reinforcing running from a column into a spread or pile footing should be detailed to be fully developed in the footing. Typically, standard hooks are used to accomplish this. Using hooks, especially if they are detailed to rest on the lower mat of footing reinforcement, facilitates construction by allowing the contractor to rest the column vertical reinforcement on the footing lower reinforcement mat, rather than having to suspend the vertical reinforcement using other methods.

Pile Embedment

When piles are used as the only supports for a pier cap (as in a pile bent), they are typically embedded sufficiently into the cap to develop a moment connection. In many cases, spiral or hoop reinforcing is provided around the embedded pile as confinement reinforcement, and in some cases a nominal mat of reinforcing is provided directly above the pile.

When piles are supporting a pile footing that support one or more columns, the piles are typically embedded at least 9" into the pile footing.

These detailing practices are usually based on owner-agency preferences.

Battered Piles

Depending on the severity of the horizontal loads on a pier, it may be necessary to provide battered piles (sometimes called brace piles) in the foundation, if the pier is pile-founded, to resist the applied horizontal forces. Typically, the number of battered piles is selected so that the sum of the horizontal components of the pile axial loads balances the net applied horizontal forces.

8.6   Single Column Pier Considerations

As mentioned above, the use of single column piers is especially prevalent for narrow bridges where there is not room for two or more columns as well as for tall piers where a single, much larger column may provide a more efficient means to resist buckling. For reference, Figure 21 and Figure 22 are provided to depict common single column pier types.

The analysis of pier caps for single column piers is relatively straightforward, but there are several key aspects that need to be considered.

Columns in single column piers do not have the benefit of frame action to help brace the columns and reduce internal moments. Also, single column piers are often used on bridges with longer spans and taller pier heights. As a result, columns in single column piers are frequently candidates for more refined column analysis methods to more accurately assess slenderness effects, as well as more complicated designs (such as hollow columns).

For grade separation bridges, Article 3.6.5.1 of the AASHTO LRFD BDS [1] requires the analysis for a vehicle collision force of 600 kips under an Extreme Event limit state if the columns are not barrier protected. The lack of redundancy in single column piers, particularly for narrow bridges with relatively small columns, may require attention from the designer to verify that a vehicle impact will not cause a collapse.

Also, in some cases, the use of post-tensioning is warranted in the pier caps of single column piers to handle the higher loads which may result from heavier, longer-span superstructures and the longer overhangs common in this pier type. Post-tensioning is also sometimes used in the columns of single column piers.

Figure 21 Single column hammerhead pier with an inverted T pier cap.

Figure 22 Single column piers with haunched pier caps and single, unsymmetrical curved columns with form liner treatments.

The detailing considerations for single column piers are fundamentally not very different than for multi-column piers in most cases. When hollow columns are used, obviously there are special detailing issues to consider. Due to the unique nature of each hollow column design, detailed discussion of appropriate detailing considerations are beyond the scope of this chapter, and designers are encouraged to consult appropriate hollow column design references.

8.7   Pile Bent Considerations

As mentioned above, a pier cap supported on multiple steel or precast concrete piles is sometimes called a "pile bent." Typically, in a pile bent, there is no distinction between the "columns" and the "foundations" – the foundations are just continuations of the piles supporting the cap, as is shown in Figure 23.

Figure 23 Pile bent with braced steel H-piles serving as both columns and foundations.

Pile bents are popular for shorter span structures where high loads and aesthetics are not a concern. Pile bents are particularly appropriate for water and wetland crossings or other long viaduct-type crossings of environmentally-sensitive areas, where extensive excavation for drilled

shafts or for pile footings is undesirable. Pile bents can be very economical in these situations due to the extreme simplicity of their construction. Once the piles are driven, the only remaining substructure work is the construction of the pier cap.

One innovative application of pile bents which has become more common in recent years is "top-down construction." For top-down construction, the structure is built span-by-span; the piles for the next bent are driven by equipment placed on a recently completed span and reaching out to the next bent location. When the piles are driven, the pier cap is placed and cured and then the next span is constructed. Once that span is complete, the pile driving equipment advances to it and reaches out to drive piles in the next bent. In this way, temporary environmental impacts are reduced to practically zero as construction equipment is located on the completed structure. In most cases, this technique is limited to very short span structures (e.g., in the 50' span range) due to practical limits on the reach of the pile driving equipment, but in those applications this method can be extremely environmentally friendly and economical. This technique has also been used in longer span applications, but generally at much greater cost.

Design of pier caps for pile bents is no different than for multi-column bents and is very straightforward. The analysis of the piles as the primary vertical supports for a pile bent is also fairly straightforward and, in some ways, is simpler than the analysis of columns and foundations for typical multi-column piers since there is no difference between the foundations and the columns – they are one in the same. However, this can also lead to complications in the analysis in some cases when pile bents are used in taller pier applications. Since driven piles are typically fairly slender members and since there is no pile footing to provide intermediate bracing, piles in pile bents warrant careful design including judicious determination of effective length factors (K-factors) and consideration of slender column moment magnification effects. Seismic effects have been investigated in recent publications as well [33, 34].

8.8   Straddle Bent Considerations

As mentioned previously, a straddle bent is a multi-column pier in which an extremely wide column spacing is used to allow for the passage of a roadway directly below the pier, such that the pier is "straddling" the roadway below. This facilitates the passage of vehicles or trains beneath the structure, as is shown in Figure 24. Due to the unusually wide column spacing and the resulting long span length of the pier cap, straddle bent pier caps are often specialty structures such as steel box beam pier caps or post tensioned concrete pier caps.
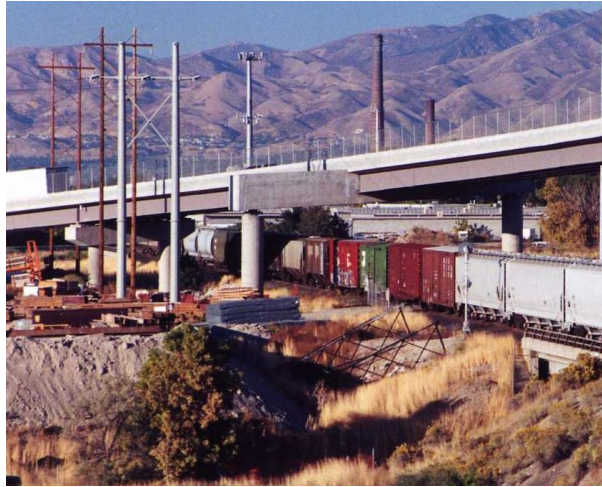
Figure 24 Fully-integral straddle bent carrying a highway bridge over a railroad.

Straddle bents may also require special design of their supports. In some cases, it may be worthwhile to provide a bearing between a straddle bent pier cap and one of its supporting columns in order to provide relief for shrinkage or thermal expansion/contraction effects.

If a steel box beam is used as the pier cap of a straddle bent, designers are advised that provisions should be made to facilitate future inspection of the interior. A single steel box straddle bent cap is considered a fracture-critical element requiring more frequent inspections, and typically steel box beam straddle bent pier caps include such features as access hatches and minimum internal dimensions which facilitate access. These should be considered early in the design process as they often control the dimensions of the steel box beam section.

Alternately, straddle bents may be designed using two or three I-shaped girders for the bent cap, which may address the issue of redundancy and result in a design which is not considered fracture-critical (depending on the owner's criteria and evaluation of the details). A recent three girder design was demonstrated to meeting current system redundancy requirements and provided an economical and constructable solution [35].

Constructability is also a serious consideration for straddle bents. Straddle bents are most often used in very congested urban settings where it proves impossible to locate piers and lower roadways separate from each other. In many of those cases, the lower roadway may already be carrying traffic and complex construction staging and traffic control may be required not only for the straddle bent construction but also for the construction of the superstructure which the straddle bent is to support.

8.9   Integral Pier Cap Considerations

It is occasionally desirable to construct pier caps integral with the superstructure. Sometimes this offers advantages in terms of structural efficiency, sometimes it offers aesthetic benefits, and sometimes it helps reduce structure depth and improve vertical clearances. Integral pier caps for

steel girder bridges have been constructed both using steel [24] and using concrete [25], the prior being depicted in Figure 25.

Figure 25 Steel plate integral pier cap for a single column pier.

Generally, each instance of the use of integral pier caps, depicted in Figure 26, is unique since there are so many project-specific variables affecting the design, including:

- The nature of the superstructure loads – An integral pier cap for a curved, continuous plate girder bridge will be subject to different loading than an integral pier cap for a straight, tub girder bridge.

- The nature of the superstructure type – Integral pier caps feature extremely complicated detailing, including a basic choice as to whether to splice the pier cap around continuous girders or to splice the girders around a continuous pier cap. Different superstructure types and sizes will suggest different preferences for integral pier cap detailing.

- The nature of the integral pier cap – A cast-in-place post-tensioned concrete integral pier cap will obviously require different detailing and shoring during construction than a steel integral pier cap, and each will carry loads in a different way and require different details for connection to the column.

- The nature of the column(s) – The location, number, size, materials, and configuration of the column or columns of an integral pier cap will suggest different preferences for type and configuration of the integral pier cap.

Figure 26 Post-tensioned concrete integral pier caps for single column piers.

As many choices as there may be for integral pier caps, there have been nearly as many solutions previously executed to some degree of success. Designers of integral pier caps are encouraged to keep an open mind regarding possible solutions and to put careful thought into layout and detailing considerations early in the design process as minor details may end up being controlling design features.

One possible solution which may eliminate many of the complications of integral pier caps is to choose to provide an expansion joint in the superstructure at the integral pier cap location, albeit at the cost of losing the benefits of girder continuity. Making the superstructure discontinuous reduces superstructure design moments to zero and may allow for the use of dapped girder ends and either an inverted-tee pier cap or possibly even a conventional pier cap which still provides the same vertical clearance benefits as an integral pier cap, but without the complicated design, detailing, and construction associated with an integral pier cap.

8.10   Steel Pier Considerations

While most piers are constructed from reinforced, prestressed or post-tensioned concrete, there are still situations calling for the use of steel for part or all of a pier. One common opportunity for using steel piers is for temporary bridge structures, such as temporary access bridges on construction sites shown in Figure 27. In these cases, contractors often prefer using steel substructures since they are usually light weight and easy to handle, relatively quick to install, and potentially reusable. Other opportunities for using steel elements in piers include long span straddle bent caps and integral pier caps.

Figure 27 Temporary work bridge with steel pipe piles, steel pier caps, and a timber deck.

When using steel piers, such as what is shown in Figure 28, care should be taken to carefully design and detail the connections. Due to the reversible nature of many substructure design loads such as wind, longitudinal braking force, thermal expansion/contraction, etc., many connections will have very high stress ranges, while simultaneously the discontinuous nature of many of the connections can potentially lead to unavoidably poor fatigue details with very low allowable stress ranges. Fatigue design can very easily be the controlling limit state in the design of steel substructures in permanent bridges.

Another serious concern when contemplating the use of steel substructures for permanent bridges is geometry control. In a concrete pier, it is fairly easy to adjust column heights, bearing seat elevations, and so on, in the field by making simple field adjustments to reinforcing and formwork. Fabricated steel components are much less forgiving when it comes to misplacements that commonly result from routine construction tolerances. Designers of steel substructures are encouraged to try to include features in their detailing which offer contractors some adjustability to deal with routine construction tolerances (for example, allowing field welding of bearing sole plates to permit adjustment of bearing locations, or allowing the use of shim plates to adjust bearing seats to achieve key elevations).

Figure 28 Steel single column hammerhead piers for a dual steel box girder bridge.

8.11   Inverted-Tee Pier Cap Considerations

An inverted-tee pier cap is a pier cap with a cross-section shaped like an upside-down capital letter T, as is depicted in Figure 29. Inverted-tee pier caps offer the advantage of very low depth below the bottom surface of superstructure girders. If dapped girder ends are used, as is shown in Figure 30, the bottom surface of the inverted-tee pier cap can be flush with the bottom surface of the girder. This low profile below the superstructure offers distinct advantages in terms of aesthetics or improvements to vertical clearances below the pier cap.

Most inverted-tee pier caps are made of concrete, sometimes post-tensioned if required to achieve wider column spacings or overhangs. The design of concrete inverted-tee pier caps is more complicated than the design of regular rectangular concrete pier caps in several important ways. First, the design of the pier cap as an inverted-tee concrete section requires some care in the determination of the concrete section bending capacity, particularly in negative moment regions of the cap. If the neutral axis of the inverted-tee in a negative moment region is in the web, the compression block forces should be appropriately adjusted.

Figure 29 Inverted T pier cap in a multi-column pier.

Second, the "ledge" or corbel of an inverted-tee pier cap should be carefully designed. This design encompasses both the ledge itself as well as the web of the inverted-tee section. Inverted-tee pier cap design should be performed using STM methods; a good example is provided in the *Design Examples for NHI Course 130126, Strut-and-Tie Modeling (STM) for Concrete Structures* [32].

Finally, column design can be significantly affected by the use of inverted-tee pier caps. Since the centerlines of bearing of two contributing spans are offset very far from the centerline of the cap, any imbalance in the loads applied by either span to the pier cap causes a significant torque on the pier cap. In cases of even moderate span imbalance, this torque may represent a significant overturning moment in the column and the pier foundations, as well as potentially

requiring design for torsion of the inverted-tee pier cap itself. Once these overturning effects have been quantified, the column design proceeds as for any other pier column.

Figure 30 Dapped girder ends rest on an inverted T pier cap in a single column pier.

8.12   Other Pier Types

As has been previously mentioned, this chapter offers only a brief overview of substructure design. For piers for instance, there is a wide range of variations on the basic common pier types mentioned here. Figure 31 shows just one example of some of the variations designers may need to deal with on a case-by-case basis. When faced with unconventional pier types, designers are encouraged to develop simple, clearly-defined load paths and to keep constructability issues in mind.

Figure 31 Multi-column piers with web walls, and single column hammerhead piers supporting a later widening.

9.0 SOFTWARE AVAILABILITY

There is a wide range of software available which can do part or all of a substructure design.

Some commercial software packages can perform many of the design steps of a bridge substructure. These types of programs can build the substructure geometry, build the superstructure geometry, calculate loads on the substructure, perform internal load analysis of the pier caps, columns, and foundations, and then size the various members and perform detailed design. However, in most cases, these programs have some limitations in terms of how complicated a design they can handle in terms of complex geometry, complicated superstructure types, complicated loading, etc. Designers are encouraged to make sure they fully understand the capabilities and, more importantly, the limitations of these "all-in-one" substructure design programs.

Substructure design can also be performed using a combination of general FEM analysis models and specific design software. Often designers will calculate many of the loads on a substructure by hand, input these loads into a simple frame model of the abutment or pier (or into a more complex model if needed), run the model to determine the internal load distribution, and then perform detailed design checks using either commercial software, home-grown spreadsheets or programs, hand calculations, or a combination thereof.

10.0 SEISMIC DESIGN

Throughout this chapter, very little mention has been made of seismic analysis of substructures for steel bridges. This deferral was intentional. The complex and highly site-specific nature of seismic analysis makes a detailed discussion of seismic analysis of substructures impossible in the limited scope of this chapter. Instead, designers are advised to consult the following sources for each specific bridge design:

- Local Owner-Agency Design Guides and Manuals – In most regions where seismic design is an issue, the local owner-agencies have adopted very specific policies regarding how seismic design of substructures is to be performed. The Caltrans SDC (Seismic Design Criteria) is one such example [36].

- AASHTO Specifications – AASHTO LRFD BDS [1] and AASHTO *Guide Specifications for LRFD Seismic Bridge Design* [37] address seismic design of substructures; their provisions should be reviewed in conjunction with local owner-agency guidelines.

- Recent Research Papers – The field of seismic analysis, design, and detailing for bridges is constantly evolving. Designers are encouraged to review the current research for the latest developments and recommendations, in conjunction with AASHTO and local owner-agency guidelines.

- Recent Textbooks, Manuals, and Guides – There are several good manuals and textbooks that cover the basic precepts of seismic design of bridge substructures. One good guide is the Reference Manual for NHI Courses 130093 and 130093A, *LRFD Seismic Analysis and Design of Bridges* [38].

11.0   PRECAST SUBSTRUCTURES

An increasingly popular advancement in the design and construction of bridge substructures is the use of precast substructure elements. The range of possible applications of precast substructure elements is broad, and some have proposed entire prefabricated bridge systems. However, in practical terms, the most promising applications currently are precast pier caps and precast columns.

Precast substructure elements have been widely used as a tool in the Accelerated Bridge Construction (ABC) toolkit, particularly in congested urban infrastructure projects, where limiting the duration of lane closures and detours is a high priority. In those cases, the use of precast substructure elements versus cast-in-place substructure elements could save days or weeks. Other highly successful applications of precast substructures have been on large, viaduct-type bridges, either over water or land, where the repetition of identical structural elements lends itself to recovery of the initial investment in forms, etc. One such application of ABC methodologies is depicted in Figure 32, which depicts cast-in-place columns with dowels projecting up from the columns into pockets in a precast pier cap. Adjustable blocking set atop friction collars was used to temporarily support the pier cap and facilitate leveling prior to injecting grout into the dowel pockets and the gap between the columns and the pier cap.

In addition to the economic benefits of precast substructures on large projects, and the time savings in terms of reduced field construction time on urban projects, there are other benefits to precast construction, including improved quality control (due to easier control of workmanship in a controlled fabrication environment of a precast yard versus on-site construction) and reduced temporary environmental impacts (due to reduced risk of formwork failures and potential spilling of concrete during casting and reduced material handling in the field).

Figure 32 Setting of a precast pier cap.

The use of prefabricated substructures has been widely published recently, so several good references exist, including [39, 40, 41]. AASHTO has also published *LRFD Guide Specifications for Accelerated Bridge Construction* [42].

12.0 SUPERSTRUCTURE / SUBSTRUCTURE INTERACTION

In various sections of this chapter the effects of fixity on substructure loads have been mentioned briefly. A few more words on this topic are warranted in a single location to emphasize its importance.

The type of loads, magnitude of loads, and distribution of loads from the superstructure to various substructure elements is directly and significantly affected by the type of connection provided between the superstructure and the substructure. A wide variety of bearing types are available to the designer, and selection of bearing types should be given significant consideration, early in the design process, for this reason. A full discussion of bearings for steel bridges is provided in the NSBA's *Steel Bridge Design Handbook: Bearing Design* [21].

Designers are cautioned that no bearing acts "ideally" in the real world. "Sliding" bearings still have some nominal static friction force to overcome before they slide. "Flexible" elastomeric bearings still have some nominal shear stiffness and transfer horizontal loads to the substructure even if anchor bolts are not provided. Designers should account for the real-world nature of the behavior of bearings as these "nuisance" loads can represent controlling design loads in some cases such as when tall piers are used.

Designers are also reminded that bearings are typically configured only for downward acting loads, but that not all combinations of superstructure reactions result in a net downward force. The transient nature of live loads, as well as the effects of geometric conditions such as span imbalance, curvature, and skew, can result in uplift on one or more bearings at a given abutment or pier.

In such cases, it may be prudent to provide appropriate tie-downs to prevent uplift. Whether tie-downs are provided or not, designers should account for the actual loads applied to the substructure as well as the potential effects on the superstructure and on the bearings in these cases. When tie-downs are provided, designers are cautioned to design and detail them carefully so that the tie-down does not inadvertently "short circuit" a needed function of the bearing. When confronted with potential uplift and the need for tie-downs, common assumptions regarding simplified behavior and what can be safely neglected should be abandoned in favor of more rigorous and thorough consideration of the behavior of the various parts of the structure.

13.0 INNOVATIVE SUBSTRUCTURES

Substructures often require unique designs to address site-specific, and bridge-specific, goals and constraints. Innovative solutions are often required. In some cases, creative but reasonable and well-thought-out ideas can lead to good solutions.

Figure 33 Tall abutment that uses precast double-tees for the stem wall.

For example, Figure 33 shows an innovative abutment system which has been used in the Albuquerque area with some success. It is basically a full height abutment with a modular precast stem. This system has benefits typically expected from precast systems. It has been used in conjunction with a drilled shaft/pile cap system. The pile cap also acts as a leveling pad for the precast double-tee. To resist horizontal loading, the double-tee's connection to the pile cap is post-tensioned. Once the double-tees are in place and backfilled, the abutment cap is then cast on top of the double-tees. Some benefits when compared with MSE wall systems are:

- Decreased construction time.

- Minimized span length by setting the centerline of bearing just beyond the face of barrier.

- Reduced excavation by eliminating the excavation required for strap embedment.

This particular concept is not presented as a panacea, but rather as an example of an innovative, well-reasoned solution which satisfied the specific goals of a given project. Designers are encouraged to explore innovative ideas like this when such opportunities present themselves.

14.0   SUMMARY

In summary, substructure design for steel girder bridges is a broad and varied topic. This single volume in the Steel Bridge Design Handbook only scratches the surface of this subject. Designers are encouraged to consult the numerous good references that offer more detail on specific sub-topics.

There are wide ranges of options available to designers for foundations, abutments, and piers. A thorough understanding of the pros and cons of each possible candidate type will lead to better choices. Lessons of past examples, input from other local designers and from local contractors, and preferences of local owner-agencies should not be ignored.

Overly detailed analysis of bridge substructures is not encouraged. Instead, designers are advised to think about the issues involved in their design and to choose an appropriately refined level of analysis. Some of the time saved by avoiding excessively detailed structural analysis calculations can probably be better invested by considering such issues as constructability, detailing, appropriate foundation types, and superstructure-substructure interaction issues.

Overall, the old engineering adage still applies particularly well to bridge substructure design: KIS = Keep It Simple. Determine the loads from the superstructure and from other sources, and find clean, simple, robust load paths to carry those loads.

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