


Guidelines for Steel Bent Caps

G12.2–2024



AASHTO/NSBA STEEL BRIDGE COLLABORATION

American Association of State Highway and Transportation Officials

National Steel Bridge Alliance

Preface

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Foreword

This document is a comprehensive Guideline for design, detailing, and construction of steel bent caps. Steel bent caps are increasingly used, particularly in congested areas where placement of substructures is limited. Bent caps are complex due to member redundancy considerations and special attention needs to be given to design, analysis, detailing, fabrication, erection, inspection, maintenance, and load rating. To date, the onus has fallen on individual state departments of transportation (DOTs) and design firms to develop details and design practices for steel bents caps. This collaborative document alleviates that individual burden and provides professionals across the nation with the best practice information and guidance to design, detail, and construct steel bent caps successfully.

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SECTION 1—INTRODUCTION

1.1—SCOPE AND PURPOSE

This document presents the state of the art at publication with respect to design, detailing, fabrication, and construction of steel bent caps. This document is a guideline and represents steel bridge community best practices. Recommendations contained herein should not be considered as strict rules. Also, this document should be used in conjunction with the other American Association of State and Highway Transportation Officials (AASHTO)/National Steel Bridge Alliance (NSBA) Collaboration documents for further clarification on specific issues. Steel bent caps are often categorized as nonredundant steel tension members (NSTMs). This document considers nonredundant design as well as other options.

1.2—NOMENCLATURE

Collaboration—Refers to the AASHTO/NSBA Collaboration.

Connection Plate—A plate used to transfer normal and/or shear stresses from one element to another via welds or an arrangement of bolts.

Integral Bent Cap—A bent cap with longitudinal steel girders framed directly into it with a bolted splice connection which provides a full moment connection between the bent cap and longitudinal girders.

Internal Redundancy—A redundancy that exists within a primary member cross-section without load path redundancy, such that fracture of one component will not propagate through the entire member, is discoverable by the applicable inspection procedures, and will not cause a portion of or the entire bridge to collapse.

Load Path Redundancy—A redundancy that exists based on the number of primary load-carrying members between points of support, such that fracture of the cross section at one location of a member will not cause a portion of or the entire bridge to collapse. The Federal Highway Administration (FHWA) considers bridges with three or more primary load-carrying members to be load path redundant.

Non-Integral, Stacked Bent Cap—A bent cap with longitudinal steel girders supported on the top of the bent cap, with a pinned or sliding connection between the bent cap and longitudinal girders.

Non-Integral, In-Line Bent Cap—A bent cap with longitudinal steel girders connected at the same level as the bent cap and supported with a pinned or sliding connection, such as on a corbel.

Nonredundant Steel Tension Member (NSTM)—A primary steel member fully or partially in tension, and without load path redundancy, system redundancy, or internal redundancy, whose failure may cause a portion of or the entire bridge to collapse. Previously referred to as fracture critical member (FCM). At publication, AASHTO has replaced the term "fracture critical member (FCM)" with NSTM. However, publications such as the AASHTO/AWS D1.5/D1.5M *Bridge Welding Code* and ASTM A709, Standard Specification for Structural Steel for Bridges, have yet to be revised and still use the FCM term. The terms NSTM and FCM are synonymous.

Steel Bent Cap—A horizontal steel beam extending across one or more columns, also commonly called a cap beam, cross-girder, cross-beam, or pier cap.

Steel Hammerhead Pier—A steel bent cap supported by one column.

Steel Straddle Bent Cap—A steel bent cap spanning over a roadway, railroad, or other feature, supported by two or more columns; also commonly called a straddle bent, straddle cap, straddle pier, or multi-column bent cap.

System Redundancy—A redundancy that exists in a bridge system without load path redundancy, such that fracture of the cross section at one location of a primary member will not cause a portion of or all of the bridge to collapse.

1.3—ABBREVIATIONS

AASHTO—American Association of State Highway and Transportation Officials
ASR—Allowable Stress Rating
ASTM—ASTM International (formerly known as American Society for Testing and Materials)

AWS—American Welding Society
CFR—Code of Federal Regulations
CJP—complete joint penetration
CNC—computer numerical controlled
DOT—department of transportation
ERS—earthquake resisting system
ERE—earthquake resisting elements
FC—fracture control
FCP—fracture control plan
FEA—finite element analysis
FHWA—Federal Highway Administration
HLMR—high-load multi-rotational (bearing)
HPS—high-performance steel
IRM—internally redundant member
LFR—load factor rating
LRFD—Load and Resistance Factor Design
LRFD Design—refers to the *AASHTO LRFD Bridge Design Specifications*
LRFR—load and resistance factor rating
MBE—refers to AASHTO *Manual for Bridge Evaluation* (2018c)
MnDOT—Minnesota Department of Transportation
NBIS—National Bridge Inspection Standards
NCHRP—National Cooperative Highway Research Program
NSBA—National Steel Bridge Alliance
NSTM—nonredundant steel tension member
OSHA—Occupational Safety and Health Administration
SRM—system redundant member

1.4—EXISTING LITERATURE

Several other AASHTO/NSBA Collaboration documents include information pertinent to steel bent caps. AASHTO/NSBA G12.1, *Guidelines to Design for Constructability and Fabrication* includes detailing guidance for closed steel box section members, which are often used for bent caps. AASHTO/NSBA G13.1, *Guidelines for Steel Girder Bridge Analysis* includes an Article titled "Unusual Substructures and the Effect of Variable Substructure Stiffness," with information on how the flexibility of a bent cap may affect the distribution of moments and shears along the length of the superstructure. The design and analysis of integral bent caps is covered as well, but how they affect the behavior of the rest of the structure is not covered. The *LRFD Steel Bridge Fabrication Specifications* (AASHTO, 2023b) include general discussion of member geometry and steel pier caps.

State DOTs may have specific design criteria, detailing requirements, or standard details regarding steel bent caps. A state of practice review conducted by the AASHTO/NSBA Steel Bridge Collaboration in 2020 found that 13 states did (Freeman, 2020). The findings are summarized below:

- Nine states either prohibited or required approval prior to using a fracture critical steel bent cap.
- One state discouraged integral bent caps.
- Four states specified the use of infinite fatigue life (required by *LRFD Design*).
- Two states prohibited or required approval to use low fatigue category details, such as Category D, E, or E′.
- Two states required the use of high-performance steel (HPS) for bent caps.
- Two states specified a redundancy factor.
- Two states specified a minimum box height.

Some states which indicate Designers should avoid the use of steel bent caps do have them in their inventory, proving that there are some highway geometry and vertical clearance restrictions which necessitate their use.

National Cooperative Highway Research Program (NCHRP) Report 527, *Integral Steel Box-Beam Pier Caps* (Wassef et al., 2004), documents the study of various integral box-beam pier cap configurations with a focus on steel boxes framed into steel I-girders, integrally attached to a single central concrete column. These types of caps (integral with superstructure and column) are for locations of extreme clearance issues or in seismic regions where rigid connections with the substructure aid in performance. Of the responses to a survey sent to all AASHTO voting and nonvoting members and documented in NCHRP Report 527, 90 percent of integral caps were concrete. The steel bent caps discussed in NCHRP Report 527 were made integral with the concrete pier by filling a region of the steel box with concrete to engage column reinforcement. This may not be directly applicable to most bent cap configurations that readers using these guidelines are considering. However, within the design examples in NCHRP Report 527, there are discussions on the torsional rigidity of boxes with the superstructure system and how loads transfer through the box, which may be helpful to readers.

The FHWA report *Proposed LRFD Specifications for Noncomposite Steel Box-Section Members* (White, Lokhande, et al., 2019) presents the research used to develop the current non-composite steel box design provisions in the 9th and later editions of *LRFD Design*. Designers should refer to this document for more information, including examples, when using the *LRFD Design* provisions for noncomposite members for the design or rating of non-composite boxes. The updated provisions provide more refined capacities for slender and stiffened flanges than previous methods. Additionally, the report provides methods and equations for determining the resistance of boxes with stiffened geometries that fall outside the limits of the design provisions in *LRFD Design* (e.g., very slender or unequally spaced stiffeners).

Minnesota DOT (MnDOT) funded several bridge-specific reports relating to the evaluation (and in some cases, the retrofit) of existing structures with integral steel pier caps. The objective of the work was to avoid FCM designation (now called NSTM) for the structures studied. The reports illustrate very detailed finite element analysis (FEA) models created with various failures to determine if alternate load paths exist. Where no alternate load path exists, the reports suggest retrofits to supply either additional required capacity in supporting members, adding members to create alternate load paths, or adding backup elements within the member itself to achieve internal redundancy. The MnDOT Redundancy Investigation Reports are available on the NSBA Redundancy and Fracture Control website at https://www.aisc.org/nsba/design-and-estimation-resources/redundancy/.

Regarding the evaluation of existing designs and general redundancy determinations, two published documents were referenced in the MnDOT reports:

- The now-superseded FHWA Technical Memorandum “Clarification of Requirements for Fracture Critical Members” (Lwin, 2012) stated that for design and fabrication, only load path redundancy could be considered in eliminating FCMs (now called NSTMs). Only in-service bridges could utilize refined analysis to demonstrate redundancy. The memo added a designation called “System Redundant Members” (SRMs) to address structural redundancy that is demonstrated by system response. The memo required the members designated as SRMs to meet FCM material and fabrication requirements. Thus, under the memo, the designation of a member as an SRM eliminated in-service fractural critical inspections but not fracture critical fabrication and material requirements. This memo did not recognize internal redundancy for the design or evaluation of existing structures. There has been additional progress in that category since this memo was written in 2012.
- NCHRP Report 406, *Redundancy in Highway Bridge Superstructures* (Ghosn and Moses, 1998) delivers a more in-depth look at redundancy, both by evaluating the redundancy of typical bridges and supplying appropriate factors, as well as providing a step-by-step procedure to evaluate non-typical bridges. The step-by-step procedure used in the MnDOT reports in their analysis of specific structures was able to reduce the number of fracture critical bridges in their inspection inventory.

The references that are available for internal redundancy include (1) the AASHTO *Guide Specifications for Internal Redundancy of Mechanically-Fastened Built-Up Steel Members* (2018b) (*IRM Guide Specs*) and (2) NSBA *IRM Evaluator* (2023) (which is based on the *IRM Guide Specs*). Both of these references for internal redundancy supersede information from the previous two references.

An available reference for system redundancy is the AASHTO *Guide Specifications for Analysis and Identification of Fracture Critical Members and System Redundant Members* (2018a).

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SECTION 2—APPLICATION AND ALTERNATIVES

2.1—SELECTION CRITERIA

Steel bent caps are often required when roadways or railroads cross beneath the bent, typically at large skew angles when the feature carried and the feature crossed have nearly parallel alignments. The resulting long spans created by the available footprints of column footings at these large skew angles make the use of a reinforced concrete bent cap impractical. The alternative is a steel bent cap with multiple columns, shown in Figure 2.1-1(a and b) and Figure 2.1-2(a). Another variation of the steel bent cap is the single-column bent with a hammerhead bent cap configuration, also known as a steel hammerhead pier, shown in Figure 2.1-1(c) and Figure 2.1-2(b). The choice among a single column with hammerhead pier, two columns supporting a simple-span bent cap, or three or more columns supporting a continuous cap is primarily driven by two factors: (1) the width of the bridge supported above, and (2) the configuration of the feature below. A narrow (less than approximately 36 ft) bridge can be supported by a single hammerhead pier, but a wider bridge may require a simple-span steel bent cap, and very wide bridges may require a continuous steel bent cap spanning over three or more columns. Foundation conditions and the room available for foundation elements, which can be potentially limited by roadways, utilities, right-of-way, etc., may also be a factor in choosing a configuration.

(a) (b) (c)

Figure 2.1-1—Example of roadway geometry requiring a simple-span bent cap supported on two columns (a and b) and a single-column hammerhead bent cap (c)

Regarding steel bent cap column placement, the median of the feature crossed must be wide enough to accommodate column width or diameter, which can be significant for a hammerhead pier column. Otherwise, columns must be placed outside the limits of the lower roadway, including railings or barriers. Due to these placement requirements, there is often a relatively long distance between the outer columns of the bent cap and the outside of the parapets of the feature carried.

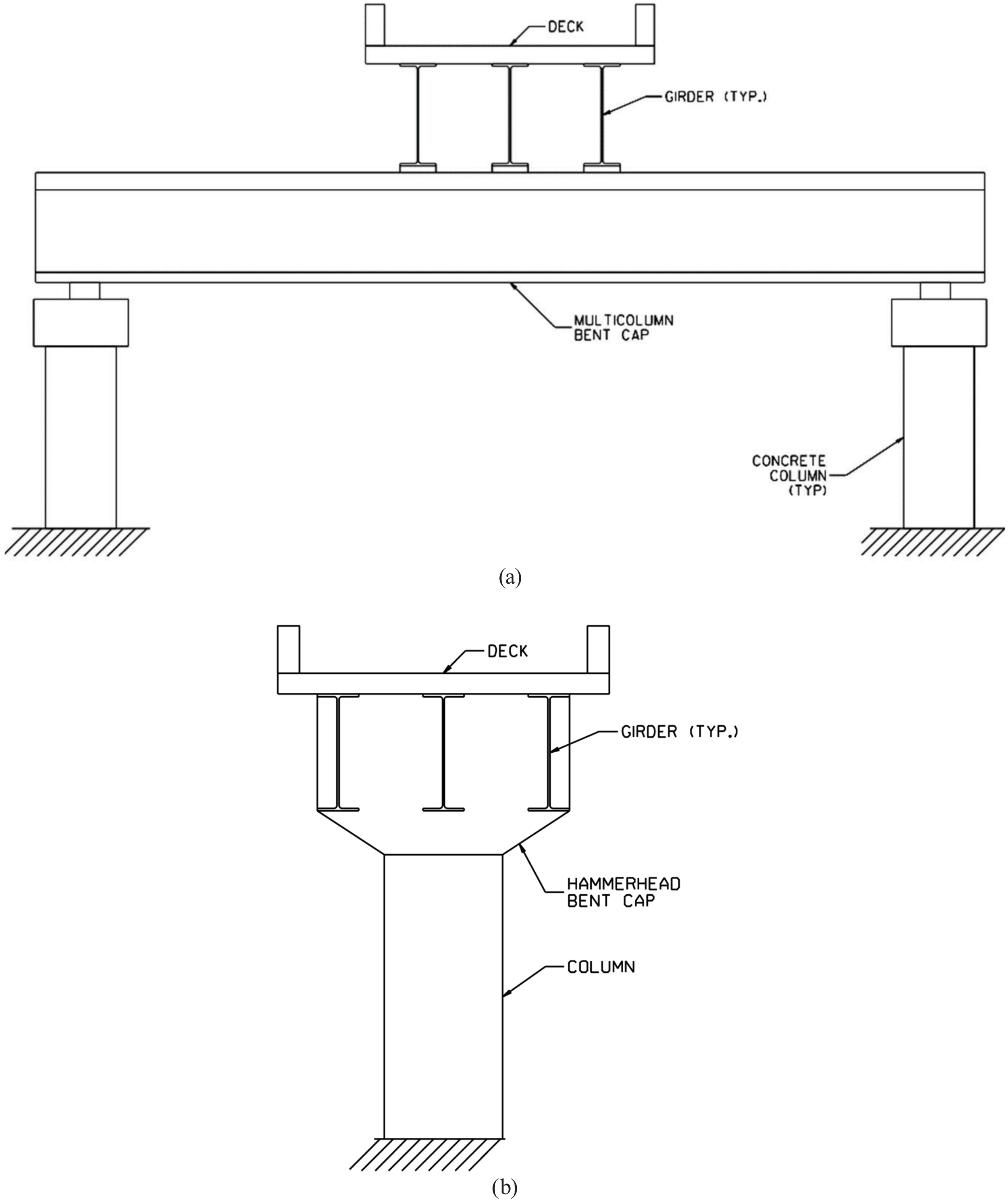


Figure 2.1-2— Diagram of non-integral straddle bent cap and hammerhead bent cap

Both the feature carried and the feature crossed can be either on a tangent or on a curved alignment, but the roadway geometry will affect the bent cap placement, skew, and span length. Bent caps placed perpendicular to the roadway centerline are preferable, as this both reduces bent cap span lengths and minimizes or eliminates some longitudinal force effects; however, skewed bent placement is sometimes unavoidable. Placing the bent cap perpendicular to the supported roadway may also simplify the connections between that structure and the bent cap, especially if the girders are framed directly into the bent cap. Refer to Article 3.6 for further discussion of skew effects.

2.1.1—Comparison to Concrete Bent Caps for Seismic Forces

Using structural steel bent caps provides a lower self-weight dead load than mildly reinforced, prestressed, or post-tensioned concrete bent caps. Further to reducing the substructure dead load demand, the lower self-weight reduces seismic forces. Compared to a steel bent cap, a solid concrete bent cap has significantly higher mass to be resisted by the substructure and foundation during a seismic event. While the support of bent caps on traditional bearings is generally the preferred arrangement, continuity of bents with the superstructure and the supporting substructure may need to be considered in high seismic regions during preliminary design. Refer to Article 3.16 for more information.

2.2—BENT CAP TYPES

As noted previously, hammerhead steel bent caps are frequently used when the feature carried is relatively narrow and there is room for a single large foundation in the feature crossed, typically in the roadway median (see Figures 2.2-1 and 2.2-2). Hammerhead columns are designed to be wide enough to provide stability. The hammerhead columns can either have vertical faces or be tapered. From left to right in Figure 2.2-1, the second steel bent cap is a non-integral hammerhead bent cap with a tapered column and the fourth bent cap is an integral hammerhead bent cap with non-tapered column. Hammerhead steel bent caps can be supported either by structural steel, as shown in Figure 2.2-2, or by concrete columns, as shown in Figure 2.2-1. If a structural steel column is used, the steel bent cap can either be bolted or welded to the structural steel column. Hammerhead steel bent caps on single concrete columns require large anchor rods and sometimes post-tensioned high-strength rods into the concrete to overcome the overturning effects of unbalanced longitudinal and transverse loading. The narrow roadways where hammerhead piers are suitable are often ramps that need to be curved to provide the proper geometric transition from one roadway to another, and these ramps are frequently superelevated. One way of providing proper roadway superelevation to a steel hammerhead bent cap is to provide a transversely beveled base plate placed between the bottom flange of the hammerhead bent cap and the top of the concrete column. An alternative method of compensating for superelevation cross slope or cross slope transitions is to taper the height of the bent cap web parallel to the deck (see Figure 2.2-2). A tapered web is not recommended for closed steel box cross-sections because of fabrication challenges.

If the steel bent cap is not a hammerhead configuration, it is a multi-column (straddle) configuration. Figure 2.2-1 shows two multi-column bent caps. From left to right in Figure 2.2-1, the first steel bent cap is non-integral and the third bent cap is integral. Typically, there are only two columns, but very heavy loading or extremely wide features carried or large skews may require three or more columns. As with hammerhead piers, multi-column steel bent caps can be supported by steel columns connected to the bent caps by bolting or welding, or the steel bent caps can be supported by concrete columns. When concrete columns are used, the steel bent cap is typically supported on bearings.

There are various options for framing longitudinal girders to bent caps. When longitudinal girders are supported on the top flange of the bent cap, bearings are used between the longitudinal girders and bent cap. Refer to Article 2.3 for more discussion on longitudinal girder framing and refer to Articles 3.15 and 4.5 for discussion regarding bearing design and detailing configurations.

Figure 2.2-1—Example steel bent caps including (from left to right) non-integral straddle, non-integral hammerhead, integral straddle, and integral hammerhead (Courtesy of New Jersey DOT)

(a)

(b)

Figure 2.2-2—Examples of integral straddle and hammerhead bent caps with tapered webs

Steel bent caps typically feature one of four different types of cross-sections: single I-section, twin I-section, triple I-section, or box section (see Figure 2.2-3). These four cross-sections can be used with both integral framing and non-integral framing. Historically, the box section has been the most typical configuration. Box sections typically exhibit higher torsional resistance than single I-sections, which is important for integral steel bent caps. Box sections are formed primarily by welding and occasionally by bolting. Single I-sections are easier to fabricate and erect compared to other configurations. Reduced redundancy should be mitigated by designing single I-sections with the appropriate redundancy load modifier as specified in *LRFD Design*. Twin I-sections are more complicated than single I-sections but are easier to fabricate and inspect than box sections. As closed members, box sections can be more resistant to pest entry and debris accumulation, but providing access for maintenance and inspection while reamining sealed can be a challenge. Twin I-sections can be connected with full-width top flange cover plates, full-width bottom flange cover plates, multiple batten plates (short plates used to connect two parallel parts of a built-up structural-steel member), vertical diaphragm plates bolted to both webs, or combinations thereof.

Connecting twin I-sections increases torsional rigidity and redundancy. However, rolled and welded twin I-section steel bents are NSTMs, as a failure in tension would probably cause a collapse of the bent due to the lack of load-path redundancy of the twin members. Steel bent caps with bolted built-up members have internal redundancy, and some multi-span structural units supported by steel bents have system redundancy, as the middle spans of multi-girder units might be able to avoid collapse if one interior bent were to fail. As an alternative to NSTM, a triple I-section configuration bent cap cross-section can be used. The triple I-section configuration is a recent development in steel bent caps. Triple I-section configurations consist of three welded I-sections connected with vertical diaphragm plates bolted to vertical connection plates that are welded to the I-section inner webs. In accordance with the FHWA Memorandum "Action: Inspection of Nonredundant Steel Tension Members" (Hartmann, 2022), the triple I-section cross-section may be considered a load path redundant design. Integral steel bent caps will require continuity diaphragms to ensure loads are transferred between girders in the event of a single I-girder fracture.

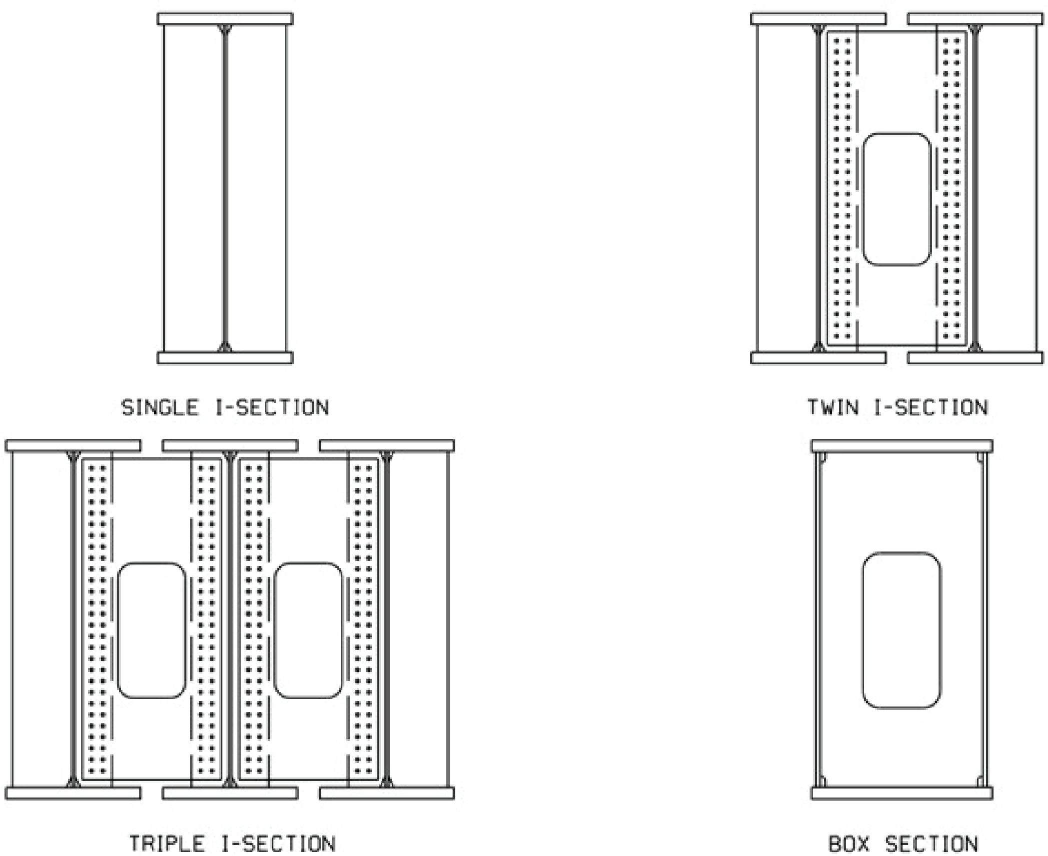


Figure 2.2-3—Steel bent cap configurations

2.3—BENT CAP AND LONGITUDINAL GIRDER FRAMING OPTIONS

Longitudinal girders supported by steel bent caps can be classified as integral or non-integral, with non-integral bent caps supporting longitudinal girders either stacked on top of (stacked configuration) or on the same level as (in-line configuration) the bent cap. The integral configuration means that the longitudinal girders frame into the outer faces of the bent cap webs, with a full moment connection. Integral bent cap framing increases the available vertical clearance below the structure, and the integral connections provide lateral–torsional bracing to the bent cap at intervals equal to the longitudinal girder spacing. The non-integral, in-line configuration similarly maximizes vertical clearance. In that configuration, the longitudinal girders frame into the outer faces of the bent cap webs without a full moment connection.

In the non-integral, stacked bent cap configuration, the longitudinal girders pass above the bent cap and bearings are placed on the top flange of the steel bent cap to support the longitudinal girders. This produces vertical loads which are generally applied at or near the horizontal center of the bent cap cross-section, reducing or eliminating eccentric loading on the bent cap. However, non-integral framing increases the moment arm of longitudinal and transverse horizontal forces above the bent cap bearings or connections; these forces may include thermal, wind, braking, or seismic.

When determining the effective unbraced length of a non-integral straddle bent cap as part of evaluating its lateral–torsional buckling capacity, Designers should consider the nature of the connection of the superstructure to the substructure and the system behavior of the bridge. For example, if sliding bearings or flexible, unrestrained elastomeric bearing pads are used to support the superstructure on a straddle bent cap, the superstructure will not function to brace the cap. In such a case, the unbraced length of the cap may be its full span length between the columns. Similarly, if fixed bearings are provided at the straddle bent cap, but the superstructure unit is not also restrained at other bents (by longitudinally restrained bearings resting on stiff piers), again the superstructure will not function to brace the cap. Conversely, if fixed bearings are provided at both the straddle bent and at another bent supporting the same continuous superstructure unit, it may be reasonable to rely on the connection of the superstructure as bracing the straddle bent cap. Structural modeling software can be used to assess the degree of bracing that other components provide.

These are just a few examples provided for illustration of the concept; each design is unique and should be evaluated on its own merits. Consider the connectivity of the bearings at the straddle bent and at other bents supporting the same continuous superstructure unit and also consider the stiffness of those other bents. Also consider the connection of the straddle bent cap to its own supporting columns to determine if the end conditions of the straddle bent cap provide torsional end restraint to the cap or not. The connectivity, framing, and stiffness of the entire structural system should be considered when determining the unbraced length of the straddle bent cap.

In many cases, the determination may be that the straddle bent cap has a relatively long unbraced length. Fortunately, the lateral–torsional buckling resistance of most straddle bent caps is enhanced by the use of closed-cell box sections or multiple I-girder sections, such that the cap has good torsional stiffness.

In all cases, the design should consider the nature of the bracing of the straddle bent cap (or lack thereof) during all stages of construction. It may be that the superstructure can function as a brace for the straddle bent cap, but only under final conditions after the end of construction.

For instance, consider the non-integral, stacked straddle bent shown in Figure 2.3.2-1. The evaluation of bracing and restraint of the straddle bent cap should include the connection of the straddle bent cap to its supporting columns, the connection of the superstructure girders to the straddle bent cap, the connection of the superstructure to the adjacent hammerhead piers, and the changes to these conditions through the various stages of construction.

There are four basic options for framing longitudinal girders to or on steel bent caps:

- Integral system.
- Stacked system.
- Corbel beam framing.
- End plate/end angle framing (not recommended).

These are described in the following Articles and in Figure 2.3-1.

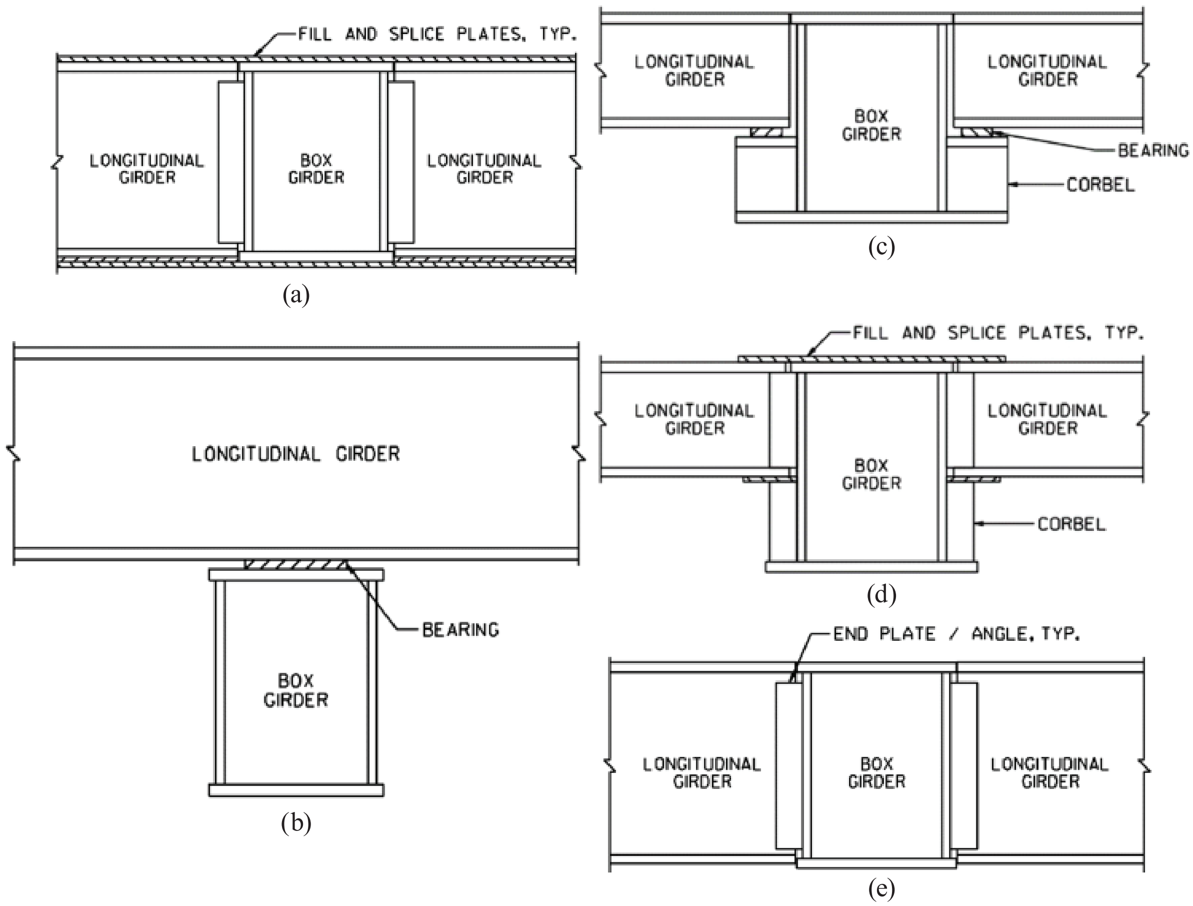


Figure 2.3-1—Longitudinal girder framing options for an (a) integral system, (b) stacked system, (c) corbel beam framing with longitudinal beams on bearings, (d) corbel beam framing with integral connection, and (e) end plate/end angle framing (not recommended)

The different bent cap framing types tie in with the topics of redundancy, stability, and bearings. Each configuration has distinctive bearing requirements, and an in-depth discussion of the various bearing types and the methods of attachment to the steel bent caps is presented in Articles 3.15, 4.5, and 4.7. Redundancy and stability are discussed in detail in Articles 3.7 and 3.9, respectively.

2.3.1—Integral System

Integral bent caps consist of longitudinal girders that are rigidly connected to the bent cap with full moment connections. Box sections are typically used to resist the torsion created by unbalanced dead load or live load from the longitudinal girders. Alternatively, twin or triple I-girder sections can be used. At interior bent caps, tie plates connect the tension flanges of longitudinal girders of spans on opposite sides of the bent cap to provide structural continuity. Compression flanges of the longitudinal girders can be detailed for structural continuity either with flange continuity plates or by aligning the longitudinal girder bottom flange with the bent cap bottom flange such that they bear on one another and are attached with a tie plate. Vertical shear transfer is provided by attaching the longitudinal girder webs to the bent cap webs. This connection may be made of vertical angles or bent plates bolted to the box girder webs or by welded short steel projections (stubs) attached to the bent cap. Whichever is used, internal diaphragms aligned with the longitudinal girder webs should be provided inside the bent cap. Integral bent caps provide three benefits:

- The longitudinal girder connections provide lateral–torsional bracing to the bent caps.

- Connecting the longitudinal girders integrally with the bent cap provides structural continuity for the superstructure.
- The shallowness of integral bent caps provides additional vertical clearance for the feature passing below the bent cap.

An example of an integral system is shown in Figure 2.3.1-1, looking at an integral bent cap box cross-section, with girders framing in from both sides. Figure 2.3.1-1 shows an integral system in which the girders are relatively the same depth as the cap beam. Other alternatives have been used in practice in which the longitudinal girder depth is not as deep as the cap beam.

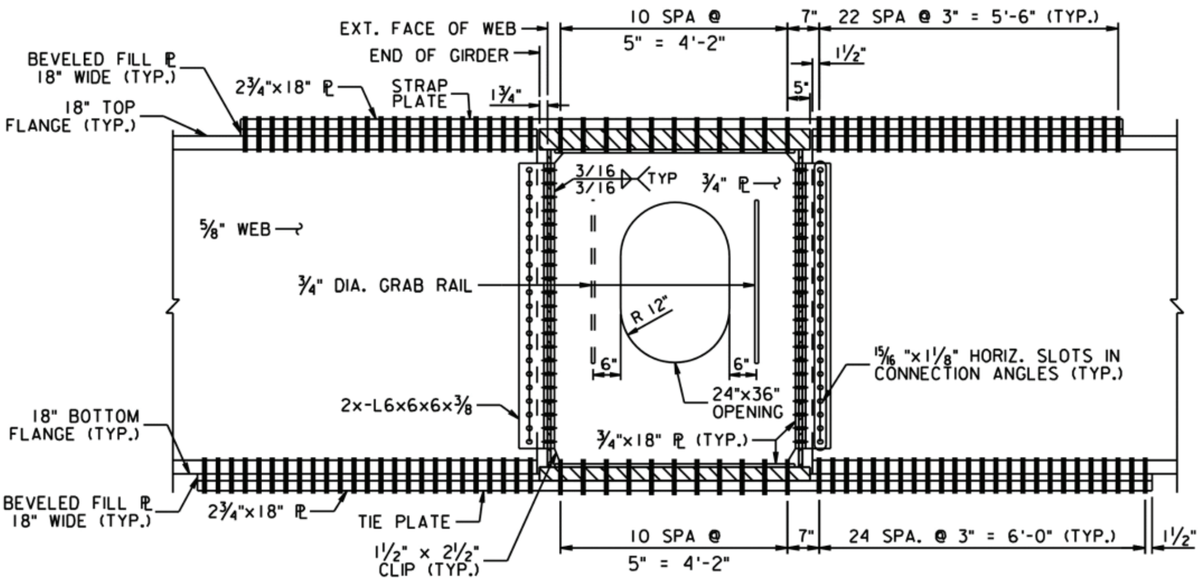


Figure 2.3.1-1—Integral connection detail

2.3.2—Stacked System

When longitudinal girders pass above bent caps, bearings are placed on the top flange of the bent caps to support the longitudinal girders. Bearings for the longitudinal girders can consist of steel-reinforced elastomeric bearings or high-load multi-rotational (HLMR) bearings. An in-depth discussion about the different bearing types and methods of attaching the bearings to steel bent caps can be found in Articles 3.15 and 4.7. The stacked system of framing longitudinal girders results in greater structure depth, so the allowed superstructure depth needs to be deeper if this approach is used. An example of a stacked system is shown in Figure 2.3.2-1. Advantages and disadvantages of stacked systems include the following:

- The stacked system provides for a simpler connection design and construction than the integral system.
- Fabrication, erection, and construction are simpler and faster compared to the integral system, leading to time savings and potential cost savings.
- More vertical separation between the carried roadway and the crossed roadway, or other feature, is required for stacked systems than for in-line systems.
- Depending on the connectivity (bearing details) and articulation of the supported superstructure (connection to other bents supporting the same unit, and their stiffness), the superstructure may or may not provide lateral–torsional buckling restraint to the straddle bent cap in a stacked system. In an integral system, it is more likely that the superstructure can be counted on as a brace, or at least can be considered as contributing to the torsional stiffness of the straddle bent cap.
- More flexible geometry can be achieved since longitudinal girders can be more easily skewed in reference to the bent cap.

- Longitudinal girders may still impart lateral loads into the bent cap through bearings.

Figure 2.3.2-1—Non-integral, stacked connection

Occasionally, stacked configurations are used along with bolsters or tall bearing support plates. A bolster is a short I-section steel member that permits elevation or grade differences to be accommodated. An example is shown in Figure 2.3.2-2. They are typically used for existing pedestals, corbels, or pier seats whose elevation is not easily modified. The top of the bolster is typically welded or bolted to the bottom flange of the longitudinal girder, and the bottom of the bolster is typically welded or bolted to the sole plate of the bearing below.

Figure 2.3.2-2—Example of bolsters

2.3.3—Corbel Girder Framing

This type of framing is typically used with box-section bent caps, but can also be used with I-section bent caps. A steel corbel consists of a horizontal steel plate welded to one or more vertical plates to form a bracket. The corbel can be bolted or welded to the bent cap web (both shown in Figure 2.3.3-1). As the corbel is often located in the tension zone of the bent cap, fatigue resistance of the welded details needs to be considered. Vertical welds for corbels are typically Category C, but horizontal welds are often Category E. An alternative is a bolted connection, as shown in Figure 2.3.3-2. Advantages and disadvantages of corbel girder framing include the following:

- An in-line bent cap provides additional vertical clearance for the feature passing below the bent cap.
- Fabrication, erection, and construction are simpler and faster compared to the integral system, leading to time savings and potential cost savings.
- More bearings are required, which may result in increased construction and maintenance costs.
- An expansion joint may be needed in the deck if expansion bearings are used to support the longitudinal girders and movement between the longitudinal girders and bent cap is required by the design.
- Increased torsional loads are imparted during construction and service, and stability during construction may be more of a concern than with the stacked or integral (full-depth) systems.
- Depending on the connectivity (bearing details) and articulation of the supported superstructure (connection to other bents supporting the same unit, and their stiffness), the superstructure may or may not provide lateral–torsional buckling restraint to the straddle bent cap in a corbel-framed system. In an integral system, it is more likely that the superstructure can be counted on as a brace, or at least can be considered as contributing to the torsional stiffness of the straddle bent cap.

Figure 2.3.3-1—Examples of corbels

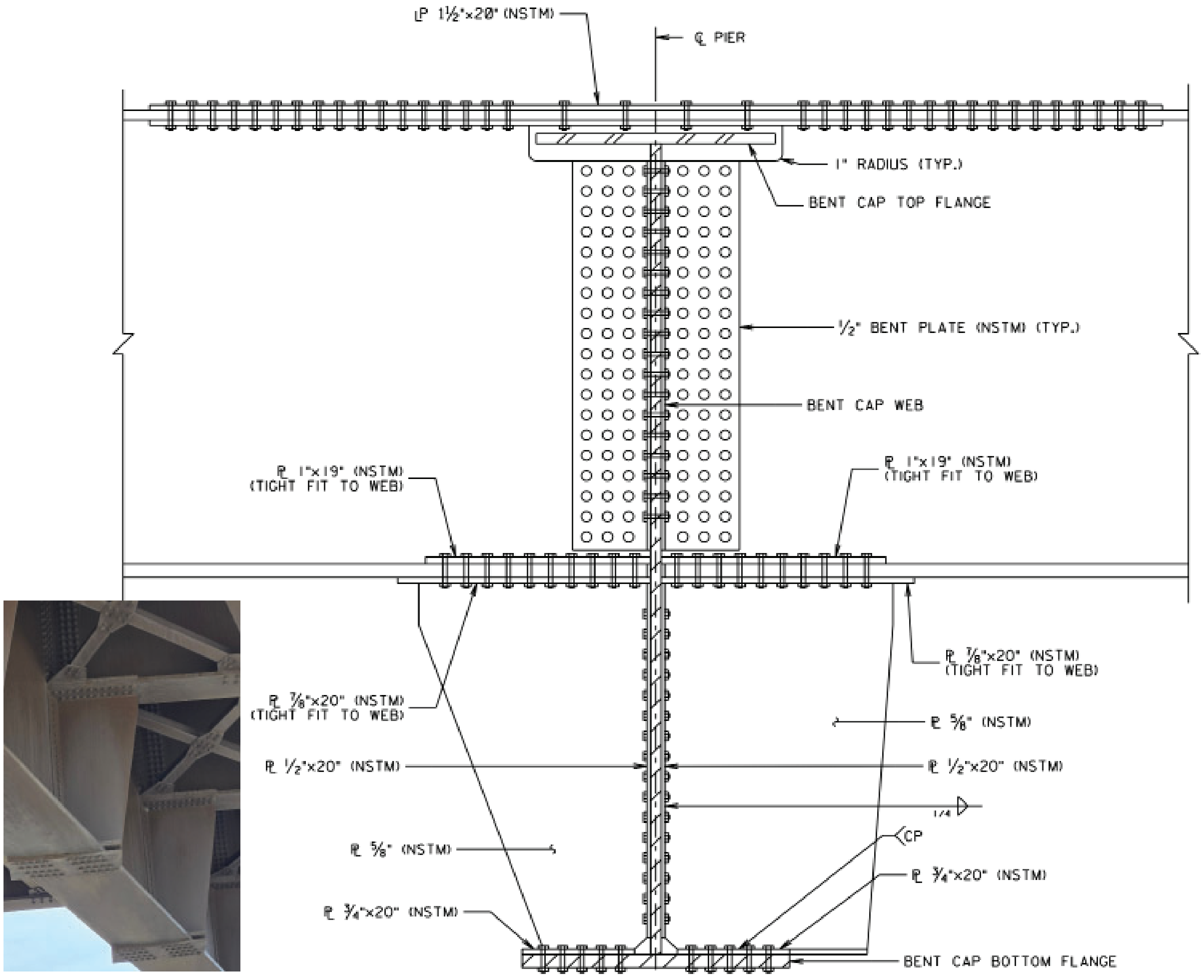


Figure 2.3.3-2—Single girder bent cap—integral, corbel connection

2.3.4—End Plate and End Angle Framing

The end plate and end angle framing system, shown in Figure 2.3-1(e), can be observed on existing bridges but is not currently being used and is not recommended for new construction. This type of longitudinal girder framing system takes one of two forms:

- One vertical plate per longitudinal girder is welded to the web of the bent cap, and the webs of the longitudinal girders are bolted to the vertical plates, or
- Connection angles or bent plates are bolted to the web of the bent cap, and the web of the longitudinal girder is bolted to the outstanding legs of the angles or bent plates.

Bolts in a single vertical plate connection to the longitudinal girder web are always in single shear, while bolts connecting pairs of connection angles or bent plates are in double shear. If connection angles or bent plates are used with a single I-section bent cap with connections on both sides of the web as shown in Figure 2.3.3-2, the bolts connected to the bent cap web are in double shear, doubling bolt capacity.

This type of framing creates simple shear connections without moment continuity at the bent caps. This advantage makes the framing method uncomplicated compared to the integral framing. However, the end plate and end angle framing does not provide system redundancy of longitudinal girders. Advantages and disadvantages of end plate and end angle framing include the following:

- This system does not have any bearings to construct or maintain.
- There is a high potential for distortion-induced fatigue cracking due to rotation at the longitudinal girder connection.
- Increased torsional loads are imparted during construction and service, and stability during construction may be more of a concern than with the stacked system.

SECTION 3—DESIGN, ANALYSIS, AND LOAD RATING CONSIDERATIONS

3.1—APPROPRIATE LEVELS AND METHODS OF ANALYSIS

The design or load rating engineer may need to complete a refined structural analysis for girder superstructure units supported on a straddle bent or integral pier cap. AASHTO/NSBA G13.1, *Guidelines for Steel Girder Bridge Analysis* is a helpful guide to steel bridge analysis. Section 1 of AASHTO/NSBA G13.1 provides a discussion of various analysis methods, and Article 3.14.3 of that same guide provides a helpful discussion of the effects of variable substructure stiffness in the context of bridges with bent caps.

The structural analysis of box-shaped sections should follow *LRFD Design* provisions for cellular and box bridges.

3.2—BRIDGE SYSTEM STIFFNESS CONSIDERATIONS

In most cases of steel bridge analysis, it is generally assumed that the vertical stiffness of all supports is infinite or at least uniform (i.e., it is implicitly assumed that all bearings at a single longitudinal support location have the same vertical stiffness). In many cases, this is a reasonable assumption. However, in some structures, the stiffness of various supports is not equal or uniform, and consideration of the vertical stiffness of various supports is necessary. This is particularly true for bridges supported on some straddle or hammerhead bent caps. Article 3.14.3 of AASHTO/NSBA G13.1 discusses system stiffness considerations for analyzing steel bridges supported on unusual substructures, including bent caps. In some cases, the effects of the configuration and stiffness of the bent cap on the behavior of the superstructure are significant.

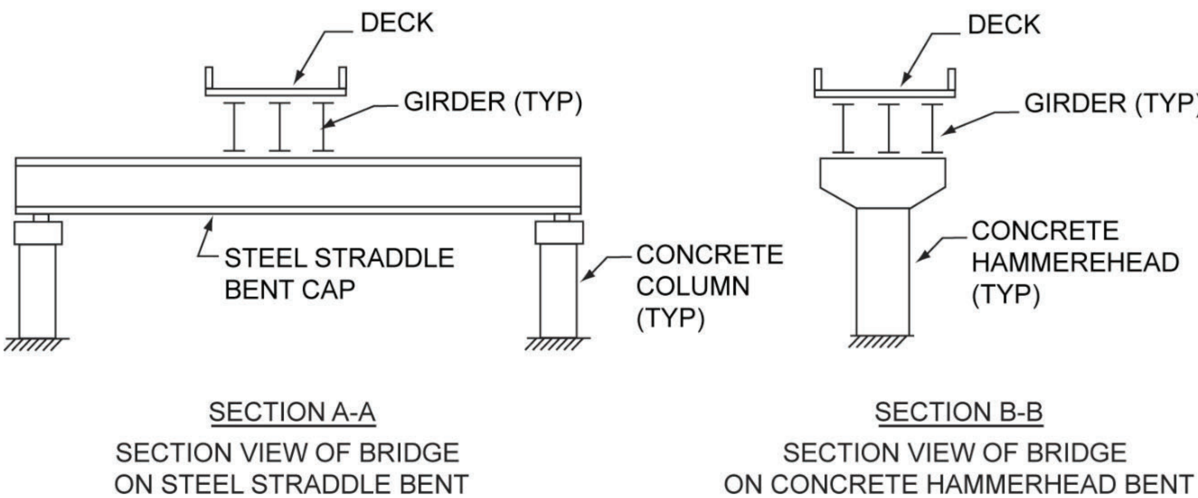


Figure 3.2-1—Girders supported by a steel straddle bent cap versus girders supported by a concrete hammerhead

For example, consider the case of a long-span steel bent cap versus a concrete single-column hammerhead bent with short, stocky overhangs as shown in Figure 3.2-1. The straddle bent cap likely provides a more flexible support than the concrete hammerhead bent. The vertical stiffness offered by the long-span steel straddle bent cap is less than that offered by the concrete hammerhead bent because the straddle bent cap has significant vertical flexibility, while the concrete hammerhead is essentially rigid in the vertical direction. If several supports of a multi-span continuous steel girder bridge are concrete hammerhead bents, with one support being a long-span steel hammerhead bent, the response of the girders to vertical loading is different than in a structure that is otherwise identical but has all concrete hammerhead bents. Figure 3.2-2 shows the elevation views of a bridge with three concrete hammerhead supports and one flexible steel bent cap support versus a bridge with four concrete hammerhead supports. The deflected shape and moment diagrams show how the superstructure response to loading may vary given different support point stiffnesses. See Figure 3.2-1 for Sections A-A and B-B.

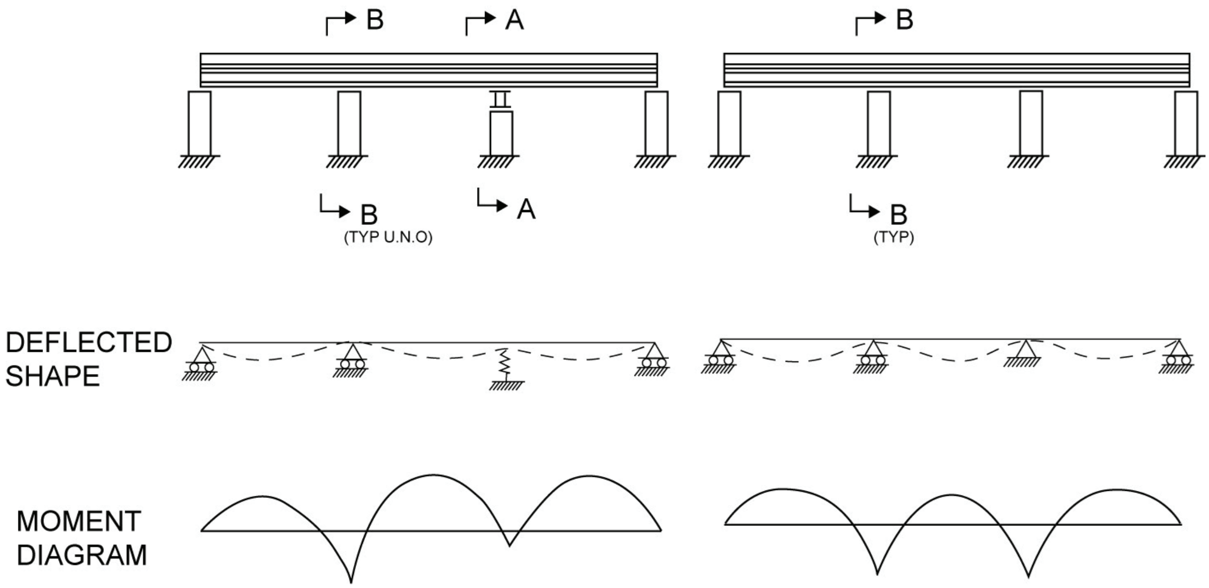


Figure 3.2-2—Bridge with three concrete hammerhead supports and one flexible steel bent cap support versus a bridge with four concrete hammerhead supports

As another example, consider a bridge with a relatively wide, multiple-girder cross-section, similarly supported at one or more bents by a steel bent cap (see Figure 3.2-3). In this case, the vertical stiffness offered by the support for the leftmost girder in the cross-section is different from that offered by the support for the rightmost girder in the cross-section. In some cases, individual girders at a given line of support may have different support stiffnesses, causing different load distribution among the girders than would be found if all girders had equally stiff supports. The difference in girder support stiffness is related to the horizontal distance between the longitudinal girder support location and the bent cap support location and the deflection of the bent cap along its length. In this case, where k_n is equal to the support stiffness of each girder, the maximum support stiffness would be at girder 6, k_6. The minimum support stiffness would be at girder 1, k_1.

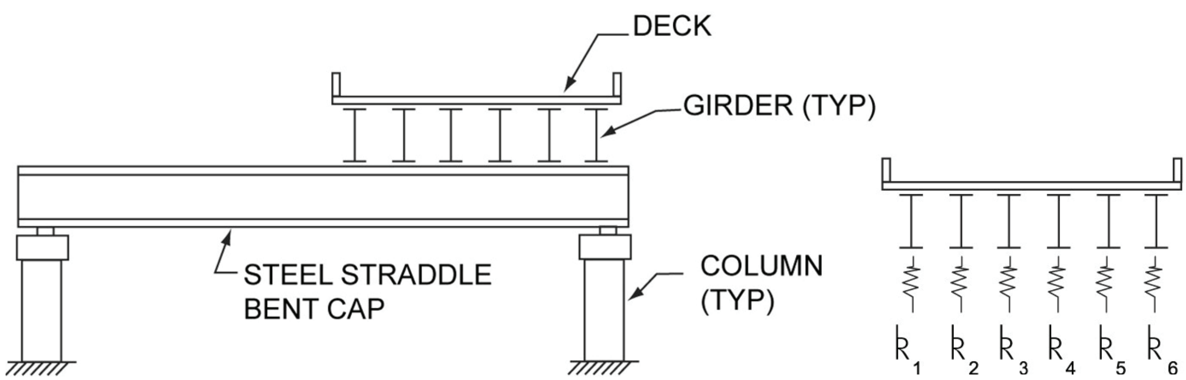


Figure 3.2-3—Individual girders at a given line of support with different support stiffness values

These types of variations in vertical support stiffness can be significant in some cases and should be considered in the analysis model. In cases such as this, it may be prudent to include part or all of the substructure elements in the superstructure model to address the effects of relative support stiffness.

While the immediately preceding discussion focuses primarily on the effects of the vertical stiffness of substructures, the lateral stiffness of substructures influences the superstructure behavior as well (e.g., the superstructure's response to thermal movements, interaction with integral abutments). In general, if these types of effects on the superstructure's behavior are of significance or of concern to the designer, consideration should be given to including representations of the stiffness of the substructures in the superstructure analysis model.

It is difficult to determine with absolute confidence the exact level of stiffness or flexibility offered by various substructure configurations. As a result, Designers are encouraged to consider a range of relative stiffness assumptions and to design the structure to accommodate any behavior within that envelope of substructure stiffness assumptions. A simple range can typically be determined by considering both the assumption of fully rigid support and the assumption of a flexible support based on the actual structure configuration.

Also of importance is whether the bent cap is non-integral or integral, as this affects the behavior (and deflections) of the bridge in general, but also, and perhaps more importantly, will affect the overall flexibility of the bridge. This will directly influence both the ease with which the erector can adjust the structural steel to achieve fit-up and the magnitude of potential locked-in stresses induced during the erection process. The use of integral bent caps, particularly in cases where the bent cap is integral with both the superstructure and the columns supporting the cap, will likely add significant stiffness to the system. This will limit the ability of the erector to adjust the structural steel to achieve fit-up.

Bearings also influence the behavior of both the superstructure and substructure of bridges supported by bent caps. In the case of a non-integral bent cap, the superstructure is generally supported by bearings resting on the bent cap. The stiffness of the bearing materials, the orientation of the bearing components, and the axes in which the bearings are free or fixed for rotation or linear movement affect the boundary conditions for the superstructure analysis model and affect the behavior of the superstructure. In most cases, the bearings allow for an ideally moment-free connection of the superstructure to the substructure. In other words, the bearings allow for free rotation of the superstructure about the transverse axis without transferring moments about the transverse axis to the bent cap. The bearings also typically allow for free rotation of individual girders about the longitudinal or tangential axis, allowing the girders to “lay over” (rotate or twist about the longitudinal axis) without directly introducing moment into the bent cap. However, this does not eliminate the effects of global overturning of the superstructure about the longitudinal axis. Many superstructures, especially horizontally curved superstructures, experience a global overturning which manifests itself in the form of differential vertical reactions across a given support. In other words, the global overturning produces vertical force couples at each support line. A straddle bent cap may be more flexible than other substructures supporting the bridge and the calculation of these reactions needs to consider the stiffness of all substructures, as discussed previously, emphasizing the need to consider the stiffness of the substructure in the superstructure model.

With regard to transverse (or radial) and longitudinal (or tangential) translational movements at a bent cap, the articulation of the bearings is critical. Bearings are sometimes designed with sliding surfaces in one or both directions (guided or non-guided bearings). In this case, the distribution of transverse (or radial) and longitudinal (or tangential) loads to the bent cap may be limited to the smaller of the calculated force effects or the static friction resistance of the bearing sliding surface. Similarly, steel-reinforced elastomeric bearings are sometimes provided at bent caps. In this case, the shear stiffness of the elastomeric bearing pad and the provision or omission of anchor rods needs to be considered. An elastomeric bearing, even without anchor rods (or with anchor rods in slotted holes in the bearing sole plates), does not provide “free” movement of the structure. The flexibility of the bearings and the flexibility of the bent cap, columns, foundations, etc. need to be considered as a system and included in the full bridge model. Longitudinal loading effects from the superstructure, whether the effects are force-driven (such as braking forces or wind forces) or displacement-driven (such as thermal expansion and contraction movements), are transferred to the bent cap. These effects produce bending moments, shears, and torsion in the bent cap and the columns supporting the bent cap.

These considerations also extends to integral bent caps. Although there are no bearings, there is still a need to accurately model the bent cap and its connection to the superstructure in the full-bridge model to correctly calculate the loading effects transmitted to the bent cap. In terms of design, “integral” can have various definitions, each with different implications. For example, a bent cap may be fully integral with the superstructure but may be provided with bearings at the columns supporting the cap, allowing rotations or translations in various axes. Alternately, the bent cap may be fully integral with the superstructure and with the columns supporting the cap, restricting rotations (or at least resisting rotations based on the stiffness of the columns).

In all cases, consideration of the bent cap stiffness, superstructure connections, bearing articulation, and bearing stiffness, in terms of both rotational and translational degrees of freedom, is necessary. Therefore, the Designers should address these considerations in an appropriately refined full-bridge model to correctly capture the effect of stiffness for both the superstructure and substructure.

3.3—MATERIAL SELECTION

The selection of the appropriate material grade is a similar process as that for the steel superstructure. Considerations include strength, fatigue, and durability. Common steel grades for steel bent caps are ASTM A709/A709M (equivalent to AASHTO M 270/M 270M) Grade 50, 50W, HPS 50W, and HPS 70W. If the optional deflection criteria of *LRFD Design* are applied, higher strength steels may not provide an economic advantage due to stiffness requirements. Additionally, the stiffness of the bent cap affects the superstructure behavior (see Article 3.2). If using weathering steel, the Designer should consider using details to protect the concrete substructure from staining resulting from water runoff of the weathering steel as shown in Figure 3.15-1 and discussed in the NSBA *Steel Bridge Design Handbook* (NSBA, 2022a) and the NSBA *Uncoated Weathering Steel Reference Guide* (NSBA, 2022b). Sacrificial thickness may need to be considered; guidance is included in NSBA *Uncoated Weathering Steel Reference Guide* (NSBA, 2022b). Whether or not weathering steel is used, it is recommended that all interior surfaces of enclosed box-shaped members be painted with a single coat of white-colored coating for inspection purposes.

3.4—GEOMETRY AND PROPORTIONS

The profile of the bent cap should be designed to accommodate the shape of the bridge cross-section and the supporting superstructure elements. Bent caps not integral with the bridge superstructure are positioned in a level or sloped orientation to accommodate the bearings of the beams or girders. If a level orientation is used, variable height sole plates can be provided to achieve the required bearing seat elevations. For a sloped orientation, beveled sole plates are used to provide a level surface for the bearing seat. Designers should include provisions to allow field adjustment of the bearing seat elevations to accommodate construction tolerances.

For bent caps integral with the bridge superstructure with a uniform cross-sloped roadway, the profile of the bent generally follows the shape of the bridge cross-section. This simplifies detailing by allowing consistent connections with longitudinal beams and keeps the top of the bent at a constant distance from the concrete deck to allow for composite action. An example of integral cap orientation is shown in Figure 3.4-1.

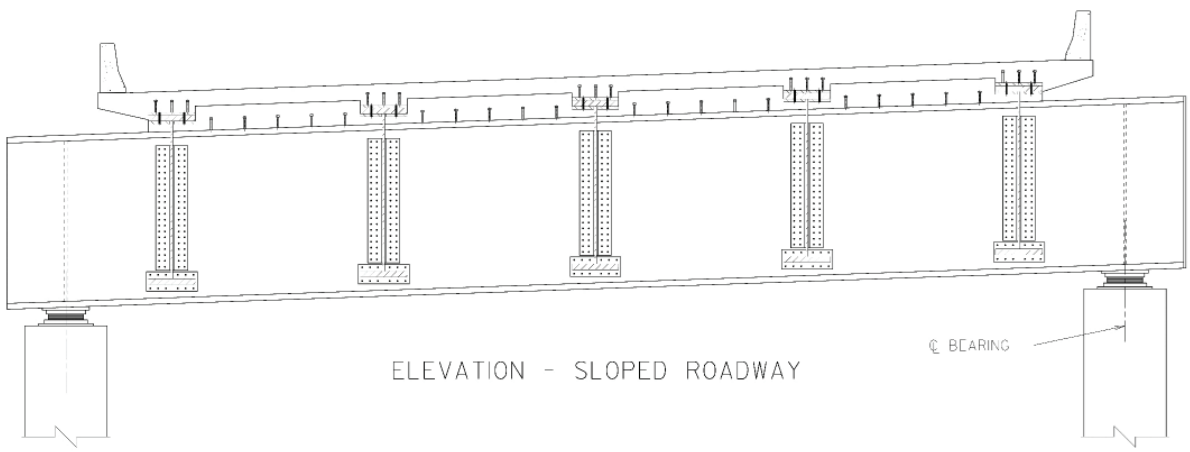


Figure 3.4-1—Integral straddle bent cap profile example

Past practice indicates that efficient straddle bent cap solutions typically have a web depth approximately equal to the bent cap span length divided by 12 (L/12). That rule of thumb can be used to obtain a starting point for design. When selecting a starting point, the longitudinal girder depth plus connection depth should also be considered, as that will impact available vertical clearance. *LRFD Design* includes design requirements for non-composite and composite boxes, including local buckling, compactness, slenderness, and stiffening requirements. In addition, *LRFD Design* and the report for *NCHRP Project 20-07, Task 415, Proposed AASHTO Guidelines for Bottom Flange Limits of Steel Box Girders* (White, Grubb, et al., 2019), cover the practical limits for unstiffened flanges, including fabrication and erection issues.

Designers should avoid the use of longitudinal web stiffeners in steel bent caps. The typical dimensions of bent cap boxes also do not lend themselves to economically stiffened flanges. In most cases, unstiffened compression flanges are the more economical and practical choice, even if they are slender. Flange stiffening has fabrication, inspection, and maintenance implications. For a continuous bent cap with a thin, stiffened compression flange, the stiffener termination in a stress reversal zone may be a Category E fatigue detail. When longitudinal stiffening is justified, the stiffeners should be plates or T-sections. Depending on internal diaphragm spacing, transverse stiffeners may increase the compression resistance of the flange. However, the addition of transverse stiffeners to increase the compression resistance of the flange is generally not economical in most bent cap applications. The connection of transverse stiffeners to webs and longitudinal stiffeners requires specific design and detailing considerations. As discussed in Article 3.17, existing bent caps may have very thin flanges with stiffening. The engineer should consider the stiffened provisions of the *LRFD Design*, Appendix E6, and the associated *FHWA Final Report for Proposed LRFD Specification for Noncomposite Steel Box-Sections* (White, Lokhande, et al., 2019).

3.5—LOADS

The same load cases as are typically involved in steel bridge design (e.g., dead load, live load, wind load, braking, thermal expansion/contraction, and seismic in some locations) should be considered for steel bent caps. See Article 3.3 in AASHTO/NSBA G13.1 for a detailed discussion of each of the applicable load cases identified in *LRFD Design*. In addition, there are several unique or unusual concepts to consider when evaluating the loading effects in bent caps:

- Horizontal forces can consist of both dead and live loads in bent cap designs, and the interaction between horizontal forces and vertical (gravity) loads should be evaluated.
- The interaction between the superstructure and the substructure should be considered, especially when integral bent caps are used; the use of a refined analysis model representing all main load-carrying bridge components is generally warranted.
- The impact of bearing articulation and stiffness on load distribution should be considered.
- The flexibility of bent caps can affect the distribution of loads in the superstructure; substructure flexibility should be considered in the superstructure model (see Articles 3.1 and 3.2) as this affects the design of both the superstructure and the substructure.
- *LRFD Design* and the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* (AASHTO, 2023a) emphasize the importance of providing a load path for seismic forces from the superstructure to the substructure. The provisions of these AASHTO documents are specifically written for "typical bridge types," which also are the basis of this document. See Article 3.16 for more detail on seismic considerations.

3.6—SKEW EFFECTS

It is generally preferable to position bent caps perpendicular to the roadway centerline of the structure, but geometric conditions sometimes make skewed orientations unavoidable. In the case of bent caps that are not integral with the bridge superstructure, the skew effects are similar to those encountered in conventional skewed pier caps, where the true orientation of lateral and longitudinal superstructure forces relative to the bent cap should be considered in the analysis. The resulting force effects should be included in the design of the bent cap and when examining the restraint conditions at the bent cap bearing points.

For integral bent caps, the effects of skew are more complex. In a skewed integral bent cap, because the primary axis of the bent cap is not perpendicular to the superstructure, the longitudinal girder bending moment is partially restrained by the primary bending stiffness of the bent cap. This can result in a significant moment transfer between the superstructure and the bent cap that should be accounted for in a structural model. Further, the resulting moment transfer should be considered in the detailing of the connection between the superstructure and the bent cap. For example, if the flanges of the girders are not directly connected to the flanges of the bent cap, significant stresses may develop in other components within the connection. Therefore, a detailed refined analysis of the connection may be needed to ensure that an adequate load path is provided.

3.7—REDUNDANCY

Redundancy is defined in Article 6.3 of *LRFD Design* as "the quality of a bridge that enables it to perform its design function in a damaged state." There are three types of redundancy: load path redundancy, system redundancy, and internal redundancy. Load path redundancy indicates a number of load paths between points of support and FHWA has defined three or more paths as load path redundant (Hartmann, 2022). Historically, most bent caps are single members, and thus have no load path redundancy.

System redundancy can be proven using a refined analysis to demonstrate that the entire bridge will not collapse if certain members are removed. If the refined analysis demonstrates this to be true, then the member has system redundancy. This could occur due to other alternative load paths that are not typically recognized in design (e.g., load can flow through alternative paths such as barriers, membrane action in deck, floor systems, etc.). However, bent caps are typically single members and do not have viable alternative load paths. Therefore, proving system redundancy is generally a futile exercise.

Internal redundancy provides multiple parallel load paths within the member itself such that if one component were to fracture, its neighboring components could carry the additional load. An important feature of internal redundancy is the parallel elements being mechanically fastened together in the tension zones, which creates an inherent fracture resistance known as cross-boundary fracture resistance (CBFR). Thus, fracture in one component cannot propagate into an adjoining component, as could occur if the components were welded together.

3.7.1—Internal Redundancy

Bent caps are generally single members resisting flexural loads, thus they have tension somewhere in their cross-sections. As primary members without load path redundancy, they are typically deemed to be NSTMs. If internal redundancy can be proven, then the member can be classified as an internally redundant member (IRM). Bridge engineers have long recognized that increased redundancy is a good thing because it leads to a more resilient bridge. Thus, designing new bent caps as IRMs is encouraged. The AASHTO *IRM Guide Specs* provide engineers an AASHTO-approved approach to reclassify NSTMs as IRMs or design a new member to be an IRM. The often-cited advantage that comes with IRM designation is reduced in-service inspection burden as compared to NSTM inspections. See Article 6.1 for more discussion on NSTM and IRM inspections and requirements thereof.

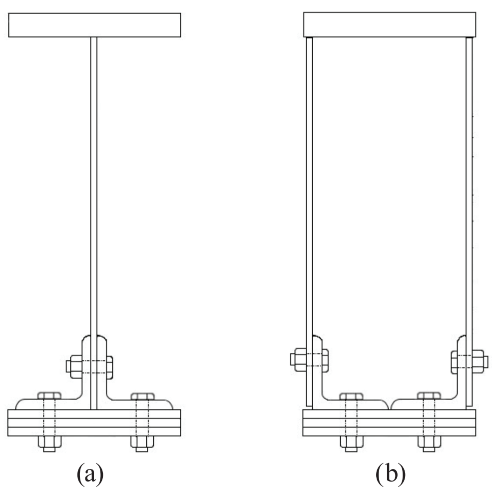


Figure 3.7.1-1—Typical built-up bottom flange of (a) an I-section and (b) a box-section internally redundant member

Figure 3.7.1-1, excerpted from the *IRM Guide Specs*, illustrates the concept of providing internal redundancy in the bottom tension flange of I- and box-shaped cross-sections. The *IRM Guide Specs* outline the following simple rules for achieving internal redundancy, such as:

- Components in tension are mechanically fastened (not welded) together.

- Elements are required to be proportioned such that the overall cross-section is made from a greater number of small cross-sectional area elements versus a small number of large cross-sectional area elements.
- A new load case is defined for checking the member in the "faulted" state. This was calibrated such that the member need not survive indefinitely in such a condition, but only to the next inspection cycle when the severed component can be detected.
- Stresses are computed on the net and gross section of the faulted member utilizing amplification factors that account for stress redistribution after the component is assumed to have severed. Stresses on the faulted gross section are checked against a simple yield criterion, and stresses on the net section are checked against a simple tensile strength criterion.
- A fatigue check is required in the faulted condition, recognizing that in the faulted condition, stress ranges would increase. This assists in determining a special inspection interval to detect a fractured component.

Design examples for flexural and axially loaded members are provided in the *IRM Guide Specs*, and a spreadsheet is available from NSBA (IRM Evaluator, 2023) to aid in evaluating existing IRMs and designing new ones. While an IRM may have been designed as such, depending on its condition in service, it may be reclassified as an NSTM; thus, for material and fabrication, IRMs have similar requirements to NSTMs if they do not have load path redundancy. Fabrication following fracture control (FC) practices will be required.

3.7.2—Redundancy Factors for Load Rating and Evaluation

Designers should also be aware that the new member will be subject to evaluation once in service. See Article 3.17. In particular, the designer should be aware of more conservatism imposed by AASHTO in the *Manual for Bridge Evaluation* (*MBE*) for members that lack redundancy. Because the factors in the *MBE* "System Factor" sections are lower than the redundancy factors in *LRFD Design* Article 1.3.4, if an engineer has an optimized new bent cap design, it may instantly not have a sufficient operating or inventory rating. Designers should consider *MBE* system factors at the design stage of bent caps.

In *LRFD Design*, the redundancy factor, η_r is a multiplier (amplification factor) on the loading (see Articles 1.3.2 and 1.3.4 therein):

- 1.00 for conventional levels of redundancy
- 0.95 for exceptional levels of redundancy
- 1.05 for non-redundant members

Owners have various criteria for levels of redundancy; different Owners could require different factors for the same structure type.

In *MBE*, the system factor, φ_s, is a multiplier (reduction factor) on the capacity of a member. Redundancy factors vary from 0.85 to 1.00 depending on superstructure type, as shown in Table 3.7.2-1. The *MBE* "System Factor" sections indicate that system factors are applied to nonredundant superstructure systems. In the absence of guidance specific to steel bent caps, some Owners may require these system factors. The designer should only consider these MBE system factors when the steel bent cap is nonredundant.

Table 3.7.2-1—System Factor, φ_s, for Flexural and Axial Effects (Compiled from MBE "System Factor" Articles)

Superstructure Type	φs
Welded Members in Two-Girder/Truss/Arch Bridges	0.85
Riveted Members in Two-Girder/Truss/Arch Bridges	0.90
Multiple Eyebar Members in Truss Bridges	0.90
Three-Girder Bridges with Girder Spacing ≤ 6 ft	0.85
Four-Girder Bridges with Girder Spacing ≤ 4 ft	0.95
All Other Girder Bridges and Slab bridges	1.00
Floorbeams with Spacing >12 ft and Noncontinuous Stringers	0.85
Redundant Stringer Subsystems between Floorbeams	1.00

Without redundancy built into the steel bent cap, the redundancy factor, η_r, for the design would be 1.05, and the system factor, φ_s, for ratings would be 0.85. With redundancy built in by some means (either internal redundancy or load path redundancy), both of these factors could be revised to 0.90 or 1.00.

Of the choices available, welded two-girder bridges, three-girder bridges, and floor-beams with spacing greater than 12 feet all have system factors of 0.85. As bent caps typically have one, two, or three parallel members, and as no system has a factor less than 0.85, a system factor of 0.85 may be considered appropriate. If a bent cap design is subjected to mostly dead and live load forces, then an efficient design with a demand-to-capacity ratio near 1.00 may not have a sufficient load rating capacity because the 0.85 system factor will reduce the recognized capacity in a load rating analysis.

3.8—FATIGUE AND FRACTURE DESIGN AND DETAILS

As described in Article 3.7, steel bent caps are primary flexural members with portions in tension. As such, steel in portions of, or all of the member must exceed designated Charpy V-Notch (CVN) impact energy levels required by ASTM A709. Most bent caps are also not load path redundant (NSTM or IRM), which, for fabrication, means the required impact energy level and testing frequency is increased and there are extra prohibitions against weld repairs at the mill. Any welding and other fabrication would have to follow the fracture control plan (FCP) requirements of AASHTO/AWS D1.5/D1.5M *Bridge Welding Code*. The collection of requirements for such members is called FC practice. Designers should designate members, or zones of members, as NSTM or IRM. While a bent cap may be classified as an IRM, Owners should recognize it could become an NSTM sometime in service (Hartmann, 2022). Failure to properly maintain the condition of the bridge, or significant inspection findings left unrepaired, may cause an IRM classification to be changed to NSTM. Furthermore, engineers who perform future rating or rehabilitation work on the bridge will need to know which members are not load path redundant and require redundancy or system factors.

3.8.1—Denoting NSTMs or IRMs on Contract Drawings

NSTM or IRM portions of members, or components of members, should be designated on plans to inform Fabricators of the material requirements for the steel, and any further enhancements required by the FCP in terms of welding, inspection, non-destructive testing, and repair. Designating NSTM or IRM portions of members, such as flanges, on the contract plans is usually a simple exercise, but there are options on how to designate the web. In some cases, the neutral axis can be close to the underside of the deck or in the deck, such that essentially the entire cross-section would be subjected to tensile stresses and should be designated as an NSTM or IRM. The neutral axis location is based on several assumptions that have significant variations affecting the neutral axis location, including minimum material properties of the concrete deck, assumed effective flange width of the concrete deck, and transition from non-composite to composite sections. If the neutral axis is within the web of the bent cap, the best practice is to designate which regions of the web are subject to tension. There are significant fabrication implications associated with designating the entire web as an NSTM or IRM, as that requires some attachments such as the compression flange and weld between the compression flange and web to be fabricated using FC practices. The FHWA *Bridge Welding Reference Manual* (2019 with September 2020 Errata) contains recommendations on how to designate fracture-critical (now NSTM) zones on steel bridge members. Guidance is also available in the NSBA white paper based on the National Bridge Inspection Standards (NBIS), "Implementation of Redundancy Terms under 2022 NBIS" (Connor et al., 2023), for use of the newer redundancy-related terminology.

There are various ways to denote members without load path redundancy in the design plans. Guidance is provided below, but in all cases, follow the required denotation method of the Owner, when applicable.

- In the general notes, mention that there are NSTMs or IRMs, and that these members need to be fabricated to FC practices.
    - Example General Note 1: *Portions of bent caps at Piers 2 and 5 are designated as NSTMs. All members, or portions thereof, identified on the design drawings as NSTMs shall be fabricated in accordance with FC practice.*
    - Example General Note 2: *Portions of bent caps at Piers 2 and 5 are designated as IRMs. All members, or portions thereof, identified on the design drawings as IRMs shall be fabricated in accordance with FC practice.*
- On the drawing sheet where the detailing of the NSTM (or IRM) is given, a note should call out the NSTM requirements. If the member cannot be clearly denoted as an NSTM only through the use of a note and it requires further clarification, add the NSTM denotation through the use of hatching or a legend, as shown in Figure 3.8.1-1.
    - Example Drawing Note 1: *The tension flanges and hatched portions of the web of the structural steel bent caps are NSTMs. Fabricate top flange plate and hatched portion of the web plate in accordance with FC practice.*
    - Example Drawing Note 2: *Bent caps are IRMs. Fabricate top flange plate and hatched portion of the web plate in accordance with FC practice.*

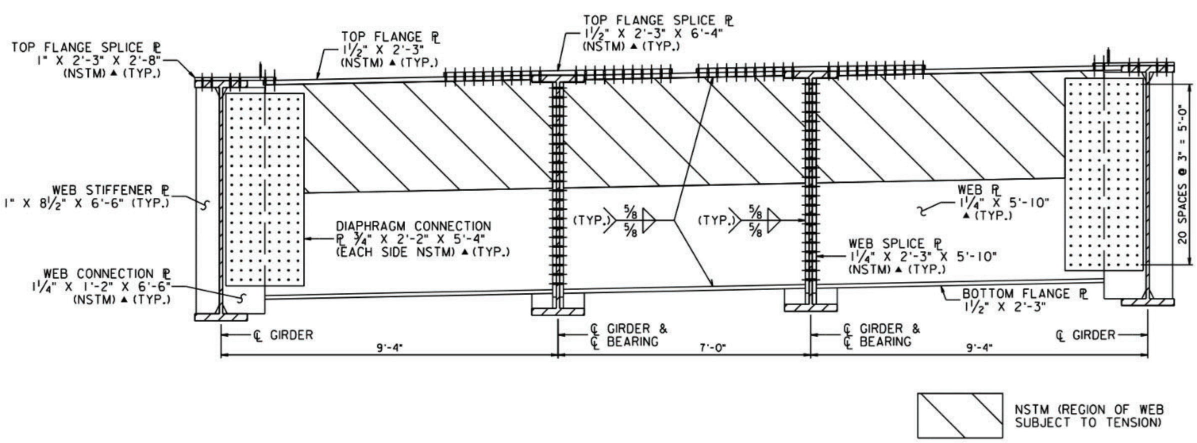


Figure 3.8.1-1—Example for denoting NSTMs on a design plan set

The steel bent cap shown in Figure 3.8.1-1 is supported by bearings below the two interior girders, and cantilevers to the exterior two girders. Any positive moment generated between the two interior girders is overcome by the dead load negative moment due to the cantilever load. Therefore, the entire length of the member is in negative moment.

3.8.2—Fatigue Design for NSTMs

In general, Designers have the option to design for either finite or infinite fatigue life for redundant tension members. For NSTMs and IRMs, *LRFD Design* and *IRM Guide Specs* mandate infinite fatigue life for design.

DOTs may have requirements that discourage or prohibit welding details that have a low fatigue detail category (i.e., Category D, E, and E′). Applicable fatigue detail categories are directly dependent on the bent cap geometry and connectivity with the longitudinal girders. Common locations where special detailing is required to improve the fatigue category include the longitudinal girder flange to bent cap stub and longitudinal attachments for drainage or other miscellaneous items. For the longitudinal girder flange to bent cap stub detail, a radius can be added to the stub (see Figure 3.8.2-1) to achieve a Fatigue Category C detail or better. For longitudinal attachments for drainage or other miscellaneous items, fully tensioned bolted connections as shown in Figure 3.8.2-2 are recommended.

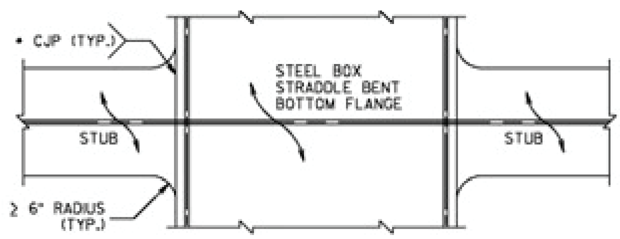


Figure 3.8.2-1—Longitudinal girder flange to bent cap stub

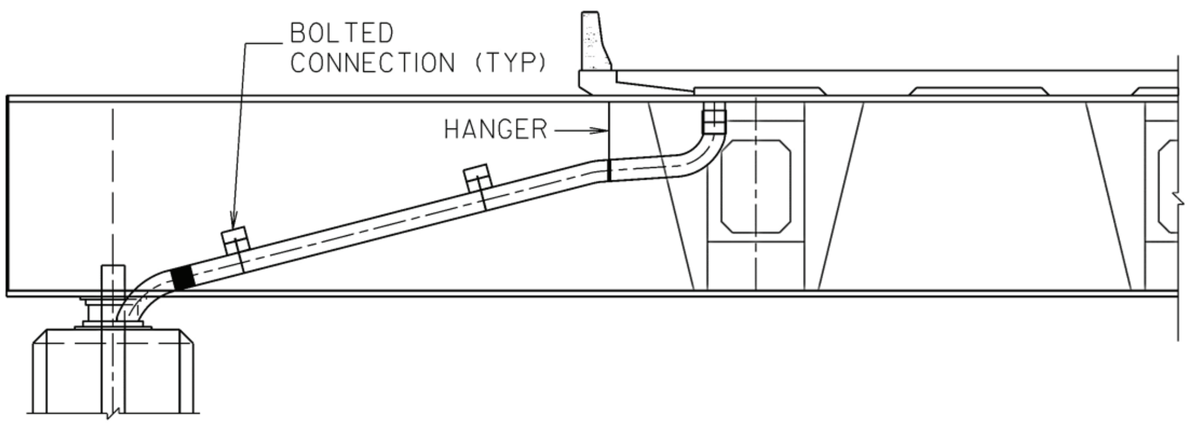


Figure 3.8.2-2—Attachments for drainage or other miscellaneous items

3.9—STABILITY AND TORSION

Two directions of global stability are essential for steel bent cap design: global transverse stability and global longitudinal stability. Global transverse stability relates to forces that cause rotation about the longitudinal axis of the bridge carried by the bent cap. Global longitudinal stability relates to forces that cause rotation about the longitudinal axis of the bent cap itself. Global longitudinal stability is critical for all types of steel bent caps, but global transverse stability typically only needs to be checked for single-column piers, such as hammerhead piers. During the design and detailing of a steel bent cap, global stability and the need to form a resisting mechanism based on chosen fixity conditions should be considered. The two directions of global stability are discussed in following Articles, followed by a discussion of member and local stability.

3.9.1—Global Transverse Stability

Hammerhead steel bent caps (steel bent caps on single columns) are configured such that the bent cap and the roadway carried by the bent cap are wider than the column or column head below the bent cap. This configuration creates a condition in which unbalanced live loading from trucks, lane loading, wind loading, or seismic loading, as shown in Figure 3.9.1-1(a), can create a force couple at the top of the column. This force couple pushes one end of the bent cap upward and the other end downward, causing a rotation about the longitudinal axis of the bridge, as shown in Figure 3.9.1-1(b). This would occur for any of the four types of longitudinal girder framing options. Integrally framed girders would provide some resistance to this overturning through span continuity, but their resistance should not be counted on for design. For a steel bent cap on a steel column, resistance to overturning can be provided by bolting or welding the bent cap to the column or by fabricating the bent cap monolithically with the steel column. For steel bent caps on concrete columns, large anchor rods and sometimes post-tensioned high-strength rods are embedded into the column to resist the overturning force couple. Where anchor rods provide stability, it is preferable to have redundancy in the number of anchor rods.

It is crucial to follow the load path downward to the footing to ensure stability and strength. It is also critical to consider the effect of transverse overturning during various construction stages—there may be an unbalanced dead load due to longitudinal girder placement or a deck pouring sequence. During these construction stages, post-tensioning of high-strength rods (if applicable) may not yet have occurred. The need for temporary supports to resist transverse instability during construction should be investigated. This should be checked during design considering any erection methods and noted in the design plans, if applicable. Ultimately it is the responsibility of the contractor and erector and their construction engineer to investigate stability during all phases of their proposed erection scheme. In Figure 3.9.1-2, transverse stability is provided by temporary shoring towers during construction, while the final bridge incorporates a wide, hinged bearing at the top of the single permanent concrete column.

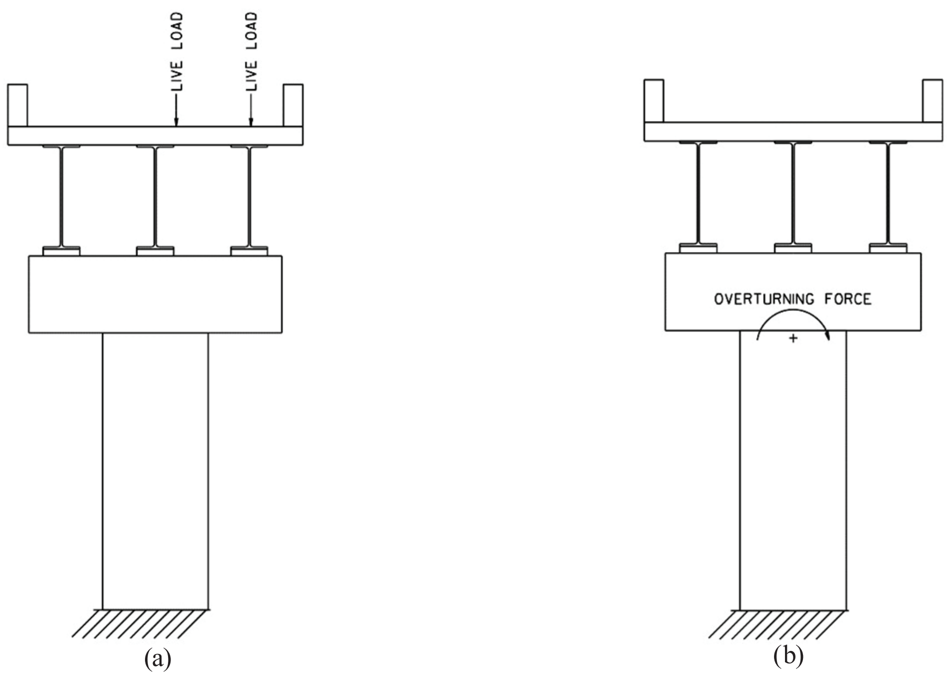


Figure 3.9.1-1—Illustration of global transverse overturning force

Figure 3.9.1-2—Hinged hammerhead steel bent cap

3.9.2—Global Longitudinal Stability and Torsion

The design of steel bent caps requires that global longitudinal stability, or rotation about an axis along the bent cap, be checked and satisfied. Rotation can be caused by loading as shown in Figure 3.9.2-1. Torsional moment can be produced if the bearings provide rotational restraint. This design check is required for any of the four types of longitudinal girder framing options. Torsional moment about an axis along the centerline of the bent cap can result from:

- Unbalanced vertical reactions (i.e., heavier reactions from one or the other span supported by the bent cap), shown in Figure 3.9.2-2,
- Integral structure moments (i.e., frame action moments when the superstructure is integral with the bent cap), shown in Figure 3.9.2-3, or
- Eccentrically applied longitudinal forces (i.e., longitudinal forces applied from the superstructure to the top of the steel bent cap through bearings), shown in Figure 3.9.2-4.

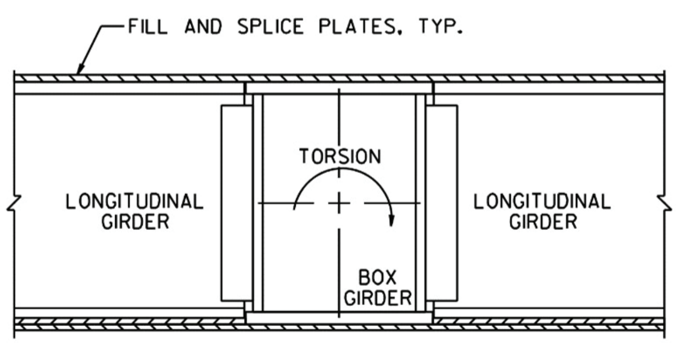


Figure 3.9.2-1—Torsion in integral bent cap system

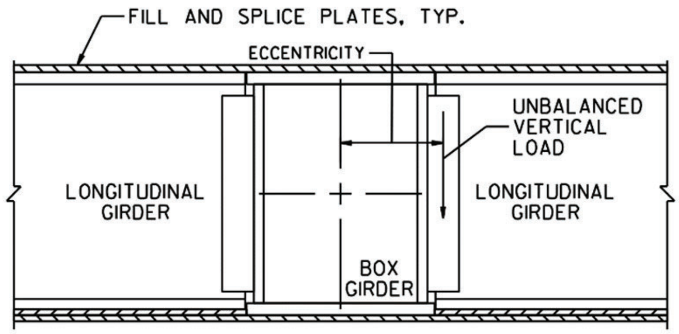


Figure 3.9.2-2—Unbalanced vertical loads in an integral bent cap system

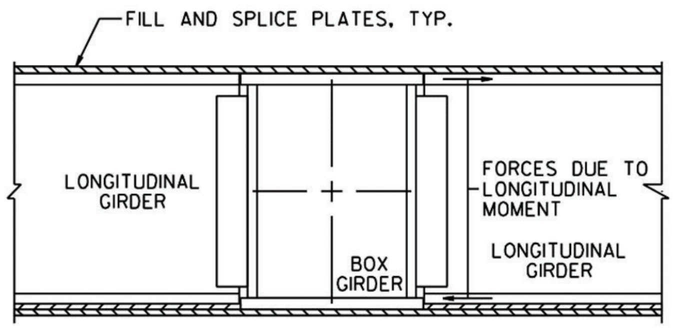


Figure 3.9.2-3—Forces due to longitudinal moment in integral bent cap system

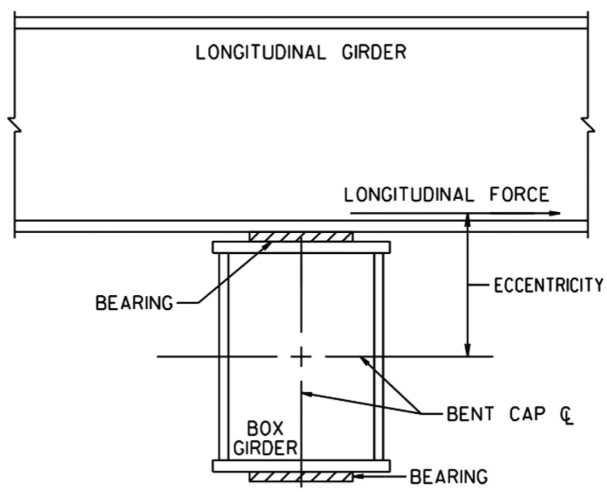


Figure 3.9.2-4—Longitudinal force in stacked bent cap system

Some means of resisting the rotation and torsional moments should be provided at the bent cap supports, to prevent an instability or roll-over failure. Individual loads that contribute to the torsional moments, and various means of resisting the torsional moment reactions at the straddle bent supports, are discussed in more detail as follows.

Placing longitudinal girder bearings on bent cap web corbels, bolting longitudinal girders to bent cap web connection angles, or having longitudinal girders integral with the bent cap may introduce significant eccentric loading on the bent cap due to unbalanced vertical loads, as shown in Figure 3.9.2-2. Unbalanced vertical loading produces torsional forces in bent caps with integral, corbel, or end plate/end angle longitudinal girder framing options. For the stacked longitudinal girder framing option, if two bearing lines are required on top of the bent cap, torsion can be induced by unbalanced, or asymmetrically located, vertical loads. Unbalanced vertical loading can occur under dead load when the longitudinal span lengths on either side of the bent caps are unequal, and by moving live loads for any span configuration.

Bent caps with integral, end plate/end angle, and sometimes corbel longitudinal girder framing are subject to torsional moment due to the bending moment in longitudinal girders and uneven shear loading at the bent cap.

Placing the longitudinal girders on bearings supported on top of the bent cap creates torsion due to the vertical eccentricity between the bent cap centroid and horizontal loads from the superstructure, as shown in Figure 3.9.2-4. Longitudinal forces are created by braking forces, wind forces, friction forces, seismic forces, thermal forces that are not relieved by deflection, or movement of bearings or other components. *LRFD Design* specifies applicable load combinations. Eccentrically applied longitudinal forces will also generate torsional moment in bent caps with corbel framing and sometimes integral and end plate/end angle framing due to the vertical difference between the longitudinal girder and bent cap cross-section centroids.

Bent caps should be designed for the combination of vertical shear and torsion from various loading sources. Torsion increases total shear on one web plate of the bent cap and decreases it on the other web plate. Moreover, as torsion is typically caused by both unbalanced dead loads and moving live loads, load combinations that include both dead and live load will control the design.

When designing and detailing steel bent caps with longitudinal girders in a stacked configuration with fixed bearings, design checks should include load combinations to maximize the effect of longitudinal forces and unbalanced vertical loads. Stability is achieved when half of the total vertical load from the superstructure is greater than the upward component of the vertical force couple resulting from the torsion. Only half of the vertical force is available to resist overturning; the half of the total vertical force applied to the web on the opposite side of the neutral axis adds to the downward component of the force couple. A free body diagram, in which half of the total vertical load from the superstructure is greater than the upward component of the vertical force couple resulting from the torsion, is shown in Figure 3.9.2-5.

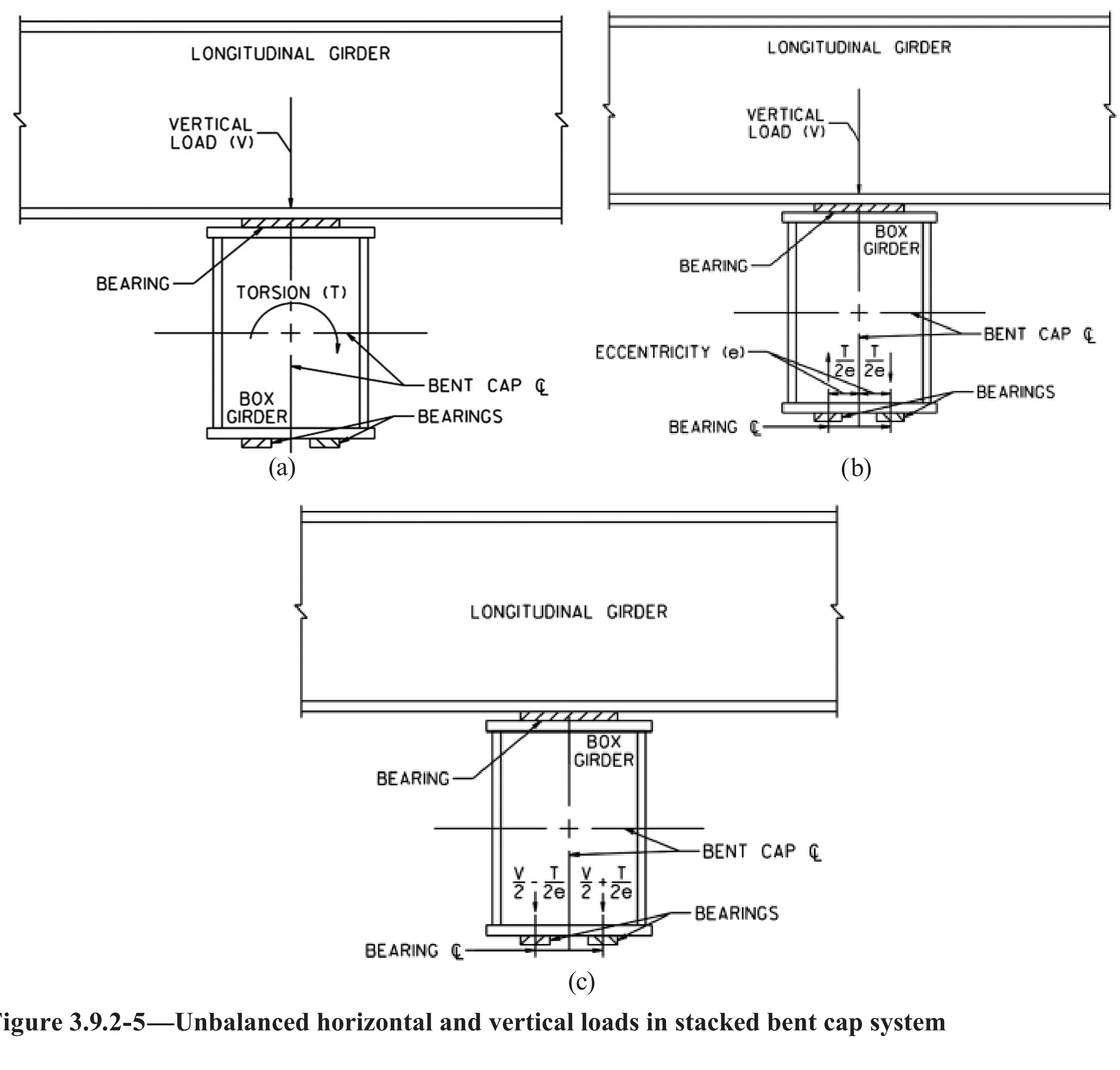


Figure 3.9.2-5—Unbalanced horizontal and vertical loads in stacked bent cap system

Bent caps supported on HLMR bearings on columns have potential for instability under longitudinal loading. Bearings at the column line are generally fixed in the global longitudinal direction and are typically narrower than the width of the bent cap box section. A force couple is created by the total horizontal load (including seismic loads, if applicable), illustrated in Figure 3.9.2-4. HLMR bearings typically have a short longitudinal dimension and thus a short moment arm to resist rotation of the bent cap. Therefore, under some load combinations, the bent cap might tip in the global longitudinal direction. See Article 3.15 for a suggested modification of the HLMR bearing sole plate that can resist tipping while not interfering with the normal operation of the bearing. Alternatively, Designers can also consider using two bearings for resisting instability, checking to confirm neither bearing of the pair are in uplift. Additional discussion of bent cap overturning in the longitudinal direction and means for resisting overturning are also covered in Article 3.15.

An alternative to resisting overturning forces is to accommodate longitudinal movements generated by the longitudinal and unbalanced vertical forces by permitting the longitudinal girder to slide in the global longitudinal direction. This can be done by using sliding or flexible expansion bearings, or by providing very flexible columns. The entire structure can deflect if the steel bent cap is attached to steel columns that are flexible in the global longitudinal direction. Existing bridges may have steel columns that deflect longitudinally to permit thermal movement, but such designs are now rare.

Integrally framed bent caps and box girder bent caps with web-supported corbels with bearings also require stability checks. Vertical forces are applied further away from the longitudinal axis of the bent cap, such as in the example shown in Figure 3.9.2-2. If bearings support the bent cap on the columns, the bearings should be designed for the force couple resulting from unbalanced forces. If there are no bearings, such as with a steel box girder anchored by rods and post-tensioned to a concrete hammerhead pier, the resulting force couple and any unbalanced vertical forces should be incorporated into the anchor rod and post-tensioning design. If steel bent caps are bolted or welded to supporting steel columns, the connection should be designed for the resulting force couple and any unbalanced vertical forces. If properly addressed during the modeling and design of a steel bent, integrally framed bents can rest on single bearings (in the longitudinal direction) without stability issues. The framing of the girders to the bent cap restrains overturning of the bent cap.

3.10—FIT CONDITION

The choice of fit condition significantly affects the detailing, fabrication, and erection of a steel bridge. Adding flexible bent caps to a bridge affects the behavior of the bridge and requires additional consideration of the choice of fit condition. The design of bridges with flexible bent caps should use a refined analysis model to accurately estimate the dead load deflections of the structure. Accurate estimates of deflections under steel self-weight and the weight of the concrete deck are critical for achieving the fit-up of a steel bridge.

The typical considerations for the choice of no-load fit, steel dead load fit, and total dead load fit still apply to bridges with flexible steel bent caps. See *Fit-up Considerations for Steel I-Girder Bridges* (Coletti et al., 2018) and NSBA Technical Resource *Skewed and Curved Steel I-Girder Bridge Fit* (Chavel et al., 2016) on this topic for more discussion of the fit conditions and the behavior of curved and skewed steel girder bridges. When reviewing those considerations, the Designers should recognize that the use of steel bent caps may affect the deflections and thus may influence the fit condition choice.

It is essential to consider the impact of the specified fit condition on any required shop assembly. Due to safety considerations, the vast majority of shop assemblies are blocked and supported in the no-load position. Thus, it is critical to recognize the vertical or rotated web position of the box girders and the longitudinal girders in such an assembly and avoid any force-fitting of components in a fit verification assembly. For the connection of longitudinal girders to an integral steel bent cap, where blocking or shoring is expected, assume no-load fit (and specify in the contract documents). As the longitudinal girders are connected to the bent cap in the field, the bent cap is increasingly loaded and approaches steel dead-load deflection profile. Cross-frames may be detailed to a different fit condition than the bent cap.

3.11—CAMBER

Geometry control is critical for the fit-up and performance of both the longitudinal girders and the bent caps. For a typical multi-column concrete bent cap type, the bent caps are relatively stiff with closely spaced columns that minimize the vertical displacement of the bent cap and girder ends. For a straddle bent cap, the distance between the columns can be much longer than in a typical concrete bent cap design. The longer span between the columns results in a noticeable deflection of the straddle bent cap, which ultimately leads to a support settlement condition of the bearings of the longitudinal girders. The magnitude of the resulting girder deflections is dependent on each girder support location on the bent cap (see Article 3.2 for discussion on system stiffness considerations). An example of the deflected shape of a straddle bent cap is shown in Figure 3.11-1. This behavior occurs for any vertical load, including dead load and live load. The support settlement condition should be investigated for redistribution of moments in the longitudinal girders in the positive and negative moment regions as described in Article 3.2, which will affect predicted camber.

Steel bent caps and their supported longitudinal girders need to be cambered to accommodate dead load deflections. The bent cap camber should include the entire dead load deflection, as shown in Figure 3.11-2. The straddle bent cap should theoretically be a straight-line support after all dead loads are placed. The longitudinal girder camber is calculated based on a zero deflection at the straddle bent cap bearing line, as shown in Figure 3.11-2, based off of a straight-line chord between supports.

Regardless of whether the bent cap is integral or non-integral, the dead-load deflections of the bent cap should be superimposed into the longitudinal girders, as shown in Figure 3.11-3. This is always done in the fully cambered position in the shop. The bent cap dead load deflections are transmitted directly into the longitudinal girders on each span side of the bent.

If the Owner requires a blocking diagram to be shown in the design plans, the dead load deflections of the bent cap should be reflected in these blocking diagrams. This is a frequent omission on jobs with bent caps and causes approval problems when the blocking diagram included in shop drawings does not match the blocking diagram shown in the design plans. Blocking diagrams should be in the "no-load" position.

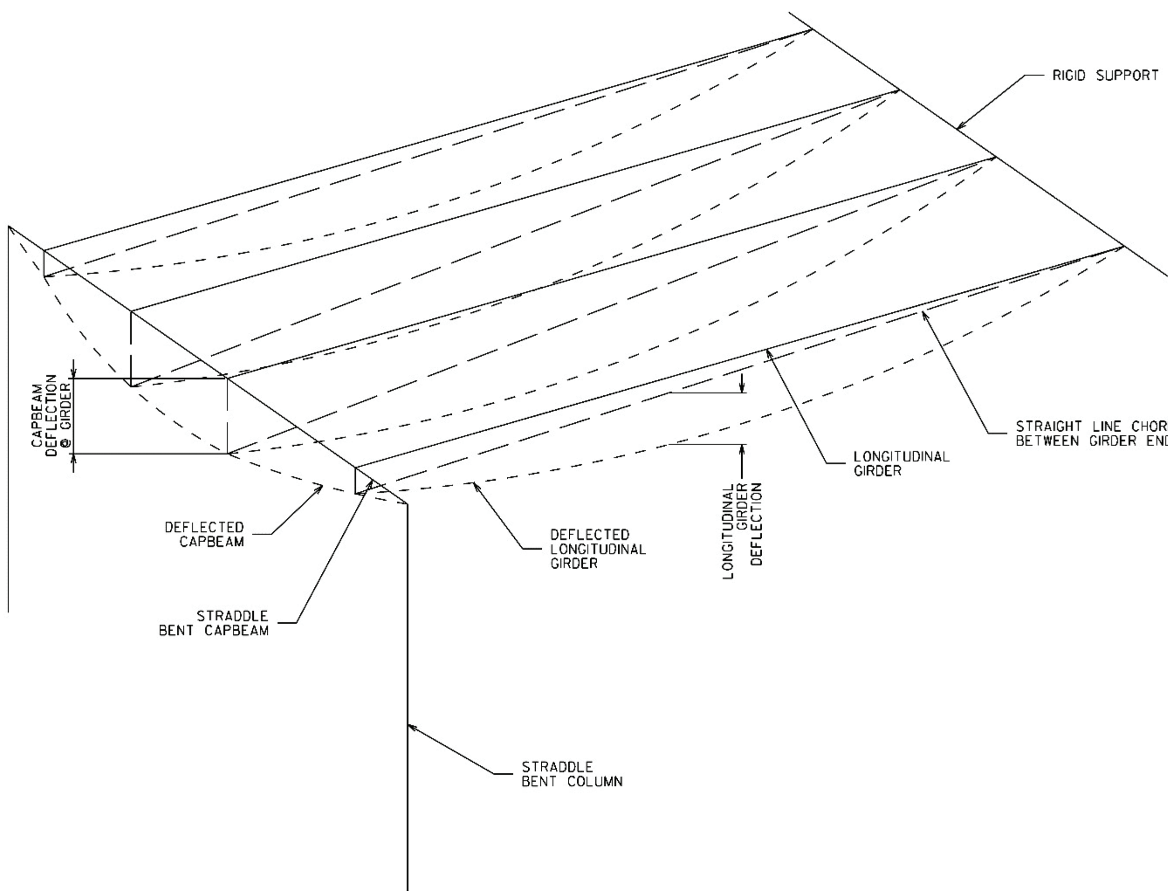


Figure 3.11-1—Example deflected shape of a bridge section

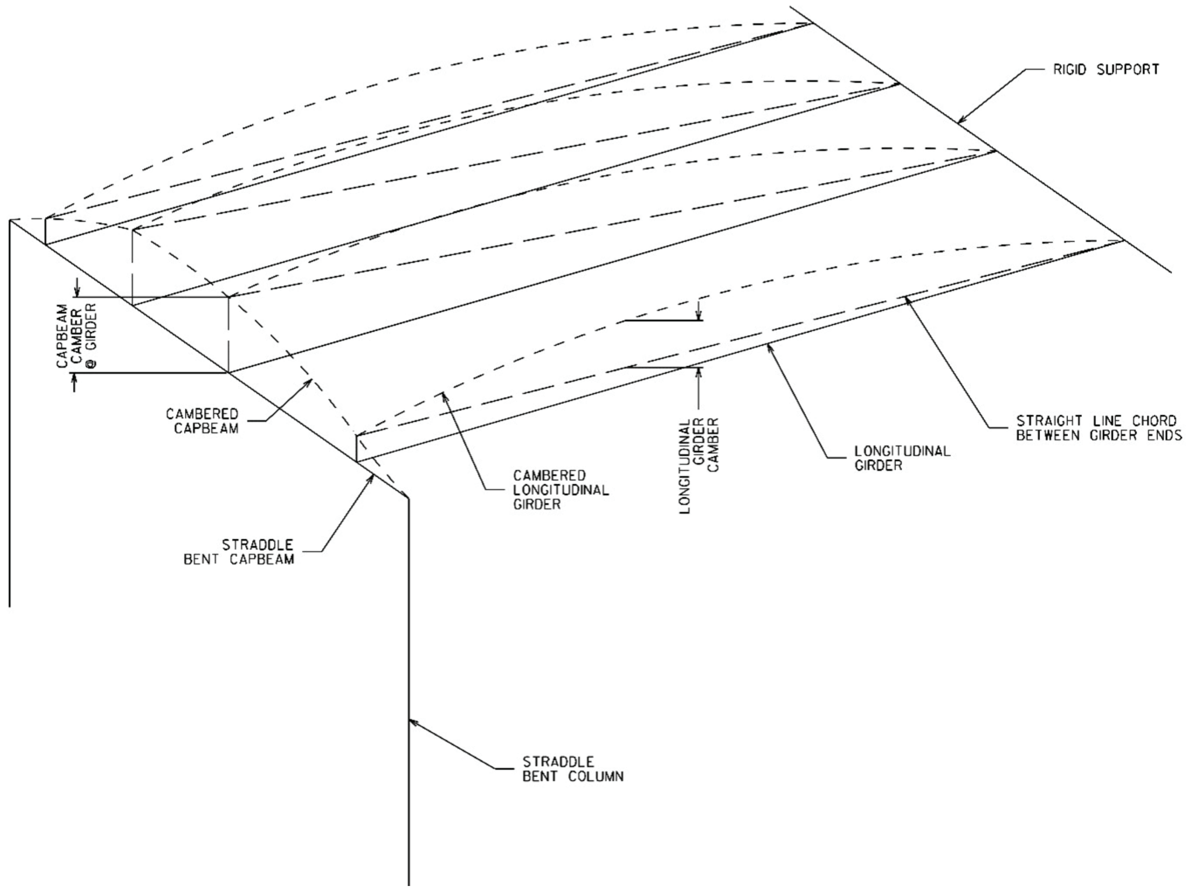
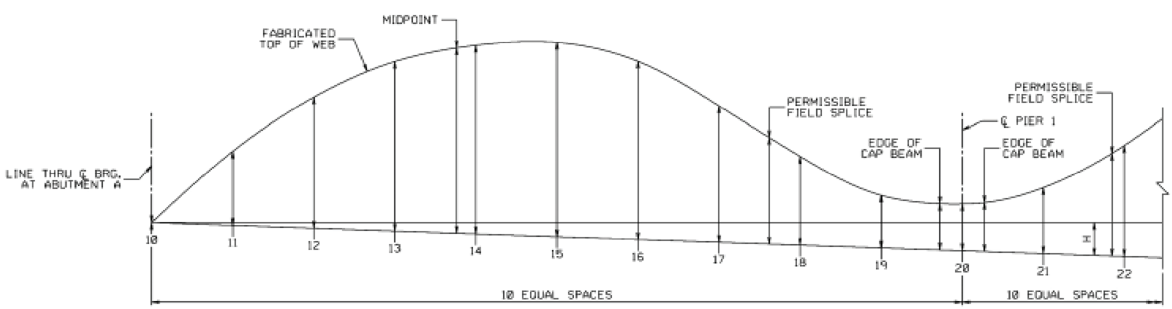


Figure 3.11-2—Example cambered shape of a bridge section



	POINT	10	11	12	13	MID	14	15	16	17	SPLICE
GIRDER 1	Δ_s	0	3/8	5/8	3/4	7/8	7/8	3/4	3/4	5/8	1/2
	Δ'_s	0	1 1/4	2 1/8	2 3/4	2 7/8	2 7/8	2 3/4	2 1/2	2	1 1/2
	Δ_c	0	1/8	1/4	1/4	1/4	1/4	1/4	1/4	1/8	1/8
	H	0	10	20	29 5/8	37 7/8	38 1/4	46 1/4	53 3/4	60 5/8	66 3/8
	TOTAL	0	11 3/4	23	33 3/8	41 7/8	42 1/4	50	57 1/4	63 3/8	68 1/2

	POINT	18	19	CAP	20	CAP	21	SPLICE	22
GIRDER 1	Δ_s	1/2	3/8	1/4	1/4	1/4	1/2	5/8	5/8
	Δ'_s	1 1/2	7/8	1/2	1/2	1/2	5/8	1	1 1/8
	Δ_c	1/8	1/8	1/8	1/8	1/8	1/8	1/8	1/4
	H	67	72 3/4	77	78	78 7/8	83 5/8	86 7/8	88 3/8
	TOTAL	69 1/8	74 1/8	77 7/8	78 7/8	79 3/4	84 7/8	88 5/8	90 3/8

DEAD LOAD DEFLECTIONS AT THIS POINT WOULD BE ZERO FOR AN INFLEXIBLE SUPPORT

Figure 3.11-3—Longitudinal girder camber diagram

3.12—LONGITUDINAL GIRDER CONNECTIONS

3.12.1—Integral Caps

The construction of integral bent caps requires field splices. There are two basic options in these situations:

1. The girders are fabricated as continuous elements running through the integral bent cap, while the bent cap itself is fabricated in field sections between the girders and spliced to the girders on-site or in the air, or

2. The integral bent cap is fabricated as a continuous element spanning between its supporting columns, while the girders are fabricated as discontinuous field sections that are spliced to either side of the bent cap.

The second option (example shown in Figure 2.3.1-1) is most common due to its better constructability. In either situation, the field splices are subject to significant moment and shear loading and possibly some level of axial loading as well (due to the longitudinal loads that may be applied to the bent cap). Load effects should be calculated using a full-bridge refined analysis model. A rigorous design of such splices should be performed. The designer should ensure the connection is designed to match the analysis with either a full moment capacity connection or a simple span connection (for the case of existing bridges with end plate/end angle longitudinal girder framing).

In most integral bent cap cases, the flanges of the superstructure girders will not be in the same plane as the bent cap flanges. This is due to the geometry of the superstructure that needs to accommodate longitudinal profiles and cross slopes. It is desirable to orient the bent cap such that the top flange of the integral bent cap is slightly below the top flanges of the superstructure girders. In this case, the superstructure girder top flanges can be spliced across the top of the bent cap.

To provide continuity for the longitudinal girder across the steel bent cap, a top tie plate is typically used. It is likely that the geometry and load conditions will only allow the use of a single continuous top plate across the connection if the steel bent cap flanges are not wide enough to develop full capacity with individual bolted connection plates to the ahead-station and back-station longitudinal girders. Doing so avoids the need to transfer flange force loading from the splice plate into the integral bent cap and then back out again through large field-bolted or field-welded connections; instead, a limited number of bolts sufficient enough to satisfy sealing pitch or similar requirements can be used for the connection of the splice plate to the top of the bent cap. Depending on the specific bridge geometry, such as the cross slope of the bent cap and profile grade of the longitudinal girders, it may be necessary to provide fill plates between the top tie plate and bent cap top flange which are beveled in both transverse and longitudinal directions.

The connection between the top tie plate and the bent cap may be altogether omitted if the detailing requirements in Article 4.5 can be met. The connection of the top flange tie plate of the superstructure girders to the top flange of the bent cap should not be necessary if a load path is provided for lateral forces such as wind and seismic loads from the superstructure deck to the bent cap. The superstructure girder splice plates can be designed to resist negative bending moments in the girders. One option for transferring lateral forces is to provide shear connectors connecting the concrete deck to both the top flanges of the superstructure girders and the bent cap top flange. Another option is to transfer lateral loads with the web connection or lateral bracing. This configuration eliminates the need for costly and difficult connections between the two flanges. Further guidance is provided in Article 4.5. The design process for the top flange tie plate is similar to other splice designs.

Web connections are made using angles or with a field splice from the web of the longitudinal girder to a vertical transverse stiffener welded to the bent cap as shown in Figure 3.12.1-1. The design of longitudinal girder to bent cap web splices for integral bent caps should consider both shear and the concurrent bending effects that may occur in the web splices due to strain compatibility. Assuming the web splices only carry web shear is generally invalid. If single-angle type connections (rolled angles or bent plates) are used for bolted web splices between the girders and the integral bent cap, prying action on the connection should be considered and can be a controlling parameter in the design of the bolted connection. In a full-moment connection, prying action is restrained by the top and bottom flange splice connections of the longitudinal girders to the bent cap.

Figure 3.12.1-1—Integral longitudinal girder connections

Geometric constraints and the associated span lengths might result in the bent cap height being deeper than the longitudinal girders, like the bridge shown in as Figure 3.12.1-1. If this is true and the design intent is to provide a moment connection, the bottom flange needs a load path to get the force into the bottom flange of the bent cap or across the bent cap to the adjacent span bottom flange. A transverse stiffener on the bent cap can be used to transfer the load from the longitudinal girder bottom flange to the bent cap bottom flange. For a continuous span, a connection plate from the bottom flange of the longitudinal girder to the web of the bent cap should be provided on both sides. This applies to a single I-section, box, or a multiple I-girder bent cap cross-section although a box or multiple I-girder bent cap would require a horizontal load path across the bent cap. For the bridge shown in Figure 3.12.1-1, the connection plate is bolted to angles on both sides of the connection and the bent cap web. Prying action would need to be considered for this detail. An alternative welded connection to the bent cap could be considered but should be investigated for fatigue. The connection plate could be welded to the ends of each of the longitudinal girders and bolted to the web of the bent cap. But that option is more challenging for fit-up and requires more attention to girder length tolerances and/or fill plates for field adjustments.

Suitable provision should be made in the design to allow for construction tolerances and to facilitate fit-up of the connections in field splices, especially in integral bent caps. The effect of any required fill plates should be considered in the design of connections. Critical considerations in the design of the longitudinal girder tie plate connection across the top of the bent cap include the interplay between the roadway grade and cross-slope, the bent cap orientation in both the vertical and horizontal planes, and the vertical position and orientation of the longitudinal girders with respect to the bent cap. The required length of bolts at these connections can become exceptional, requiring special orders with considerable purchasing lead times. Further, the detailing of the bolted connections should consider all connected components because edge distance and tightening clearances may be affected by adjacent bolts and splice plates.

Consideration should be given to the chosen fit condition when detailing the connections between the girders and the bent cap in integral bent cap designs, including consideration of the anticipated shoring that may be in place at the time of erection and assembly of the connections. Refer to Article 3.10 for more details.

3.12.2—Non-Integral Caps

For non-integral, in-line connections, steel corbels provide a seat for the longitudinal girders. The design of the corbel should include consideration of the moment caused by eccentric loading. Avoid using details with a poor fatigue performance when attaching the corbels to the steel bent cap. Attention to detail regarding the orientation of the corbel plates is crucial to prevent water from ponding and allow drainage off the corbels. Refer to Article 4.7 for further guidance.

Non-integral stacked systems typically use bearings for connection between the bent cap and longitudinal girders. Refer to Articles 3.15 and 4.7 for further guidance.

3.13—INTERNAL DIAPHRAGMS AND COMPRESSION PLATES

Internal diaphragms and compression plates play a key role in steel bent cap designs. They are used to maintain cross-sectional geometry of the bent cap, transfer forces from connected longitudinal girders across the bent cap, transfer longitudinal girder reactions to the bent cap, transfer loads between individual I-sections in multiple I-section bent caps, and resist lateral–torsional buckling of individual sections in multiple I-section bent caps.

3.13.1—Internal Diaphragms and Compression Plates in Box Section Bent Caps

If the superstructure girders are supported on top of the steel bent cap with bearings, the internal diaphragms need to be designed to transfer the superstructure girder reactions from the superstructure bearings into the two webs of the bent cap box girder. If the bent cap is integral with the superstructure girders, the internal diaphragms need to transfer the shear forces from the connecting superstructure girders to maintain shear continuity across the bent cap.

When designing an integral connection of the longitudinal girders to a bent cap, the bottom flanges of the superstructure girders are sometimes located above the bottom flange of the steel bent cap. To provide superstructure girder continuity, compression plates inside the bent cap are typically used. These plates transfer superstructure girder bottom flange compression forces through the bent cap. It is possible to cut holes through the webs of the bent cap; however, this can result in potential fatigue problems if not properly detailed. The recommended approach is to connect the bottom flange of the superstructure girders to the web of the bent cap using a compression plate that is butted against the web of the bent cap. To provide continuity through the bent cap, a similar internal butted plate detail is used to transfer force between longitudinal girder bottom flanges inside the bent cap.

The detailing of the internal diaphragms should prevent out-of-plane distortion-induced fatigue of the bent cap webs. Superstructure girder rotations can also lead to out-of-plane forces. Therefore, this connection should be treated similar to a typical girder cross-frame connection, where the diaphragms are rigidly connected to the bent cap bottom flange by welding or bolting.

3.13.2—Internal Diaphragms and Compression Plates for Multiple I-Section Bent Caps

The internal diaphragms and compression plates for multiple I-girder bent caps have design requirements similar to the box girder straddle bent caps but there are additional requirements specific to bent caps comprised of multiple I-shaped sections. Internal diaphragms transfer loads between each bent cap I-section. The size and spacing of the internal diaphragms should be set to provide relatively equal distribution of forces. In addition, internal diaphragms function as bracing to resist lateral–torsional buckling of the individual I-sections, which affects their required size and spacing.

A 3D finite element analysis is recommended to ensure each I-section carries an approximately equal load, although minor variations in load distribution to each girder are acceptable. The analysis model should include all girders and diaphragm plates along with boundary conditions consistent with the anticipated bearing configuration. Inspection access holes should be represented in the analysis model to ensure that stress concentrations in the diaphragm plates are low. Figure 3.13.2-1 shows a finite element analysis stress plot of a triple I-section bent cap. The superstructure of this bridge is supported by a single line of bearings over the middle beam of the triple I-girder cross-section. The colors in Figure 3.13.2-1 represent the stresses in the girders, which show the relatively even distribution of force between the girders.

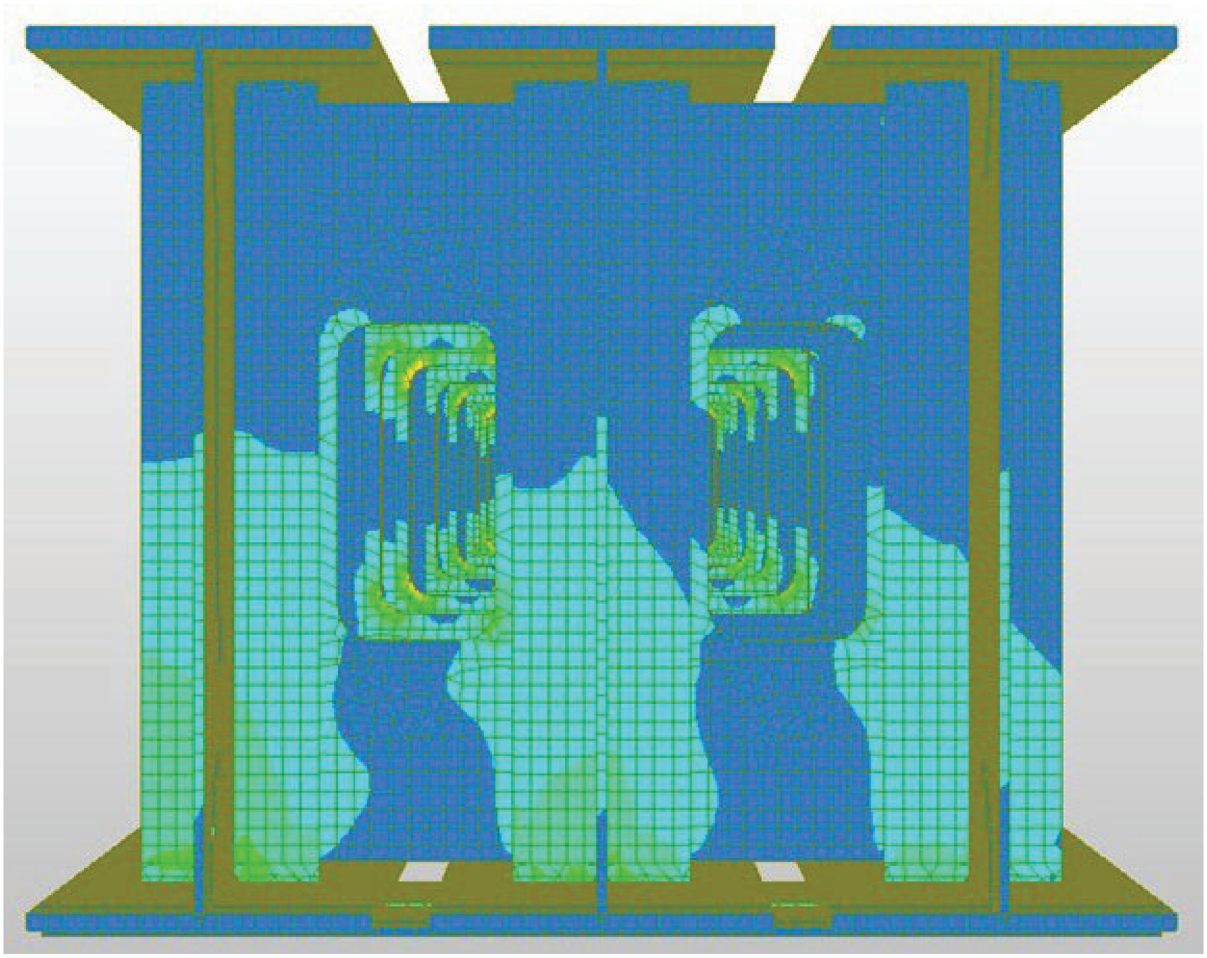


Figure 3.13.2-1—FEA stress plots of a triple I-girder straddle bent cap

3.13.3—Inspection and Construction Access Holes

Openings for inspection and construction access need to be included in the design. The Occupational Safety and Health Administration (OSHA) does not mandate a specific minimum opening dimension for worker access; however, there are industry-recommended guidelines that are generally acceptable.

A target for minimum access hole opening size is provided in AASHTO/NSBA G12.1. Bottom flange inspection access holes without permanent access ladders need to accommodate a ladder placed within the hole and need to be sized accordingly. Holes in internal diaphragms should be near the mid-depth of, and concentric with, the bent cap. The location of compression plates described in Articles 3.13.1 and 3.13.2 may affect the location of the inspection access hole. Refer to AASHTO/NSBA G1.4*, Guidelines for Design Details,* for the minimum access hole corner radius. In general, Engineers prefer larger radii and smooth transitions, whereas Inspectors prefer as large an opening as possible to facilitate access. Designers should make the access holes align across the entire bent cap to allow for safety monitoring of workers from the ends of the bent cap, and to facilitate the extrication of workers in the event of an incident.

Construction access holes to facilitate bolting are typically 8-inch-diameter circular holes. Some have cover plates which can be installed from one side.

3.14—FIELD SPLICES OF BENT CAP

The construction of bent caps, particularly integral bent caps, often involves the erection and assembly of large structural elements in the field. In some cases, the overall length or weight of non-integral bent caps may warrant shipping to the project site in field sections spliced together either on the ground or in the air. The

Engineer should investigate if length or weight limits may necessitate a field splice in the steel bent cap. The splice should be shown as optional. Weight may be a more significant consideration for steel bent caps than for longitudinal girders, and a field splice may be required if the weight exceeds 50 tons. Refer to AASHTO/NSBA G12.1 and S10.1*, Steel Bridge Erection Guide Specification*, for more information.

In most cases, bolted field splices are more economical than welded field splices since most contractors have the resources and experience to construct these splices. Welded field splices were used in the past but are no longer common as it can be difficult to keep the various planes in a bent cap aligned sufficiently for welding without external devices and the availability of skilled welders is limited. Bolted connections can be accomplished more quickly, minimizing the duration of lane closures under the bridge.

Steel bent cap field splices typically occur at locations of high moment, high shear, or both, and so the design of the connections will likely be governed by calculated design loads. That is different from longitudinal girder field splices typically located at or near inflection points.

Steel bent caps are often designed using enclosed or partially enclosed sections and field splice designs require construction access from both the outside and the inside of the cross-section. Provide adequate access for the construction workers to enter the box section and reach (and exit) the field splice location. Proportion the cross-section so construction workers have enough room to perform the required bolting or welding operations needed to accomplish the fit-up of field connections. Further requirements for safety are provided in Articles 4.6 and 5.4.

3.15—BEARING DESIGN

Bearings are typically used to connect the superstructure to a steel bent cap when a stacked system or an in-line corbel bracket system is used. See Figure 3.15-1 for an example of a stacked bent cap which requires bearings between the tub girder and triple I-section steel bent cap, as well as between the bent cap and concrete columns. These bearings should be designed to accommodate rotations and force transfer in a manner consistent with the intended articulation of the bridge. The bearing configuration selected by the designer affects the overall stability of the bent cap and provides for the transfer of force. For stacked bent caps, the designer can allow superstructure rotations to be taken by the bearings between the girder flange and the top of the bent cap or the bearings between the bent cap and the supporting columns. When the girders are made integral with the cap, a single bearing between the cap and the supporting column can be designed to accommodate superstructure rotations or the cap can be made integral with flexible columns which permit rotation.

HLMR bearings and steel laminated elastomeric pads are often used to support bent caps. Specific bearing types are covered in detail in AASHTO/NSBA G9.1, *Steel Bridge Bearing Guidelines*.

Bearing stiffeners need to be designed and detailed at each bearing. For large bearings or to accommodate significant movement, multiple stiffeners may be required.

If the superstructure girders are stacked on top of the bent cap, bearings are generally provided between the superstructure girders and the bent cap. These bearings are designed as any other superstructure bearings, which may include allowances for thermal movement and rotation of the superstructure in addition to the beam reaction forces. The bearings between the bent cap and the columns may be designed to allow thermal movement and resist superstructure reaction forces, but they also should provide torsional stability of the bent cap. This may result in multiple bearings at each column. The Designer should check for uplift of the bearings caused by applied longitudinal horizontal forces in the superstructure.

When designing and detailing steel bent caps with non-integral longitudinal girders supported on fixed bearings, the Designers should check that the bent cap is stable for horizontal loads applied in the global longitudinal direction and bent cap torsion resulting from unbalanced vertical loads. As discussed in Article 3.9, horizontal forces generate a vertical force couple that could cause instability of box girders having longitudinal girder bearings on the bent cap top flange and the bent cap supported on narrow fixed HLMR bearings. This instability could potentially result in lateral tipping of the bent cap in the global longitudinal direction. When bent cap and superstructure rotation in this direction is acceptable, consider a single bearing centered under the bent cap that allows for, and is designed for, this rotation. If no rotation is desired between the bent cap and the supporting column, two or more bearings should be used at each support to resist moment by creating a force couple.

Figure 3.15-1—Triple I-girder steel bent cap with longitudinal tub girders

Alternatively, to prevent instability resulting in tipping, a restraint (a.k.a., a "stop") can be designed to limit rotation (especially during a seismic event). These stops are in addition to, and do not affect behavior of, the primary bearing. Options exist for a moment/rigid stop. One example is to provide concrete blocks with an elastomeric pad to the outside of the primary bearing to prevent over-rotation of the bent cap. Another option is shown in Figure 3.15-2, in which a moment-rigid stop was bolted to the underside of the sole plate of the HLMR bearing. The moment-rigid stop shown is essentially an outrigger that consists of steel plates and bars and elastomeric pads, which are drilled and tapped on both sides of the bent cap box girder's bearing sole plates to limit the rotation angle of the bent cap during a seismic event. The reduced angle prevents over-rotation of the box girder about its axis (global longitudinal direction). The outrigger beam bars of the moment-rigid stop are detailed to have curved bottom surfaces so that a sharp corner is not striking the concrete seat below. Under service loads, the outrigger beam bars do not engage the column and do not interfere with the bearing's functionality; the dimensions of the moment-rigid stop components are detailed to accommodate thermal movements and service load rotations without contacting the bent cap. Using tipping restraints can change the point at which the vertical reaction is applied to the pier or column and therefore will introduce eccentricity that the pier needs to incorporate into its design.

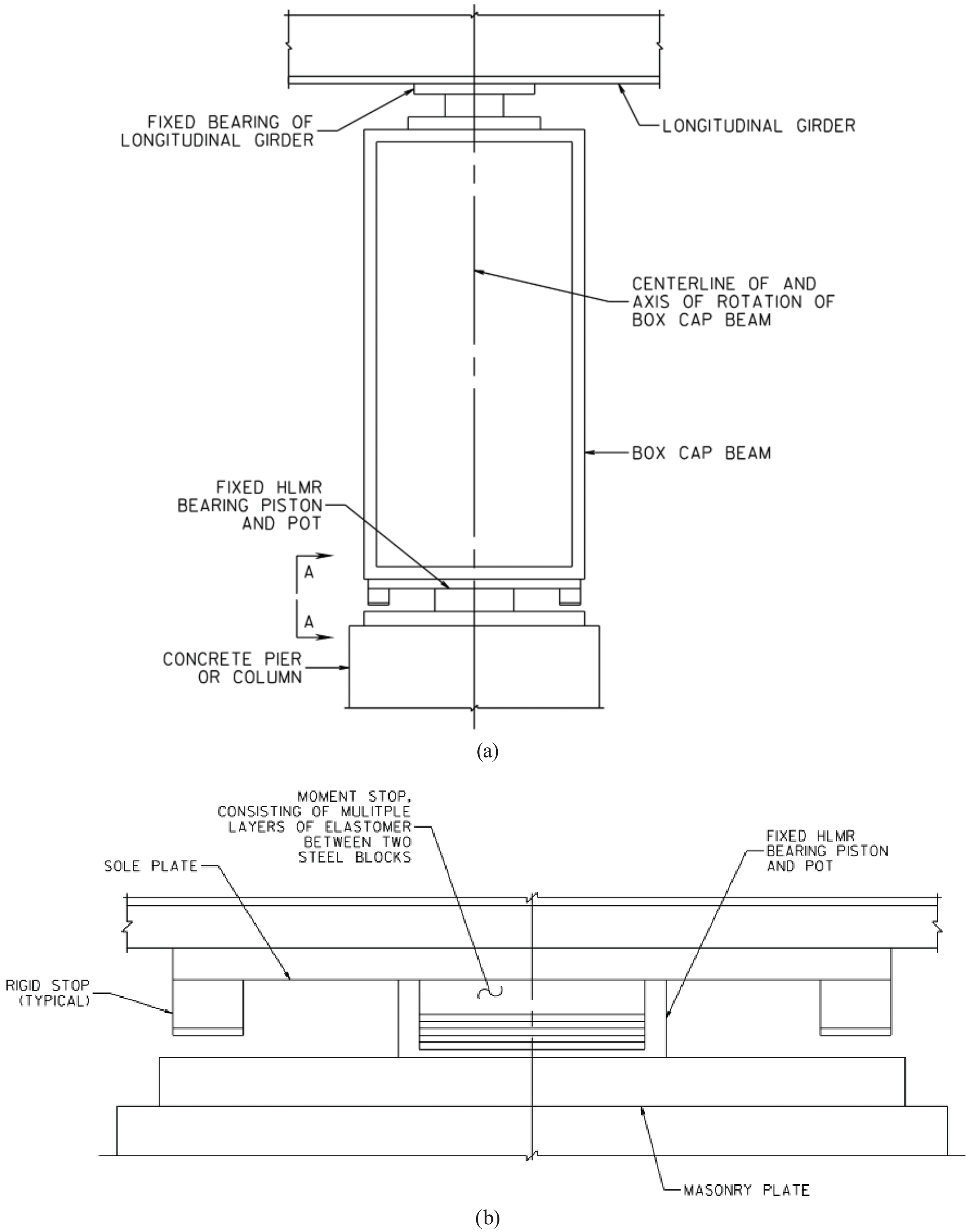


Figure 3.15-2—Rigid moment stop section view (a) and elevation view (b)

3.16—SEISMIC CONSIDERATIONS

The first steps of seismic design of these structure types are to establish:

1. The global seismic design strategy (Types 1, 2, 3)
2. The earthquake resisting system (ERS)
3. The earthquake resisting elements (ERE)

The most common global seismic design strategy is a Type 1 response, which provides a ductile substructure and an essentially elastic superstructure. The typical ERS of a Type 1 approach involves conventional plastic hinging in bent columns, which invokes detailing and design requirements as the ERE. The bent cap, whether concrete or steel, is considered a component of the essentially elastic superstructure and should be designed to remain elastic.

A Type 2 global seismic design strategy corresponds to an elastic substructure and a ductile steel superstructure. This approach is not commonly utilized and, as defined in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* (AASHTO, 2023a), requires the Owner's permission. The ERS and ERE of this system provide ductility in steel end diaphragms and cross-frames. The steel bent cap should be designed to remain elastic.

The Type 3 global seismic design strategy involves a using mechanism between the elastic superstructure and substructures, typically in the form of a seismic isolation bearing. This approach also requires Owner approval. In a Type 3 design, the bent caps are designed as elastic members and have the same design basis as the common Type 1 design approach.

So, in all cases (Type 1, 2, or 3 seismic design strategy), steel bent caps are designated as elastic steel members. Elastic steel members are designed in accordance with the steel structure provisions of *LRFD Design* for the Extreme Event I load combination. Members and connections are designed similarly for strength-based dead and live load combinations, with adjustments in the resistance factor, φ. The structural steel components provisions of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* (AASHTO, 2023a) are specific to ductile components of a Type 2 global seismic design strategy and therefore do not apply to steel bent caps.

The design of elastic steel members is always force-based. While displacement-based design is invoked by the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* (AASHTO, 2023a), the evaluation of displacements is specific to the ductile members of the ERS. Depending on the bridge's Seismic Design Category, which is a measure of the seismic hazard level as a function of spectral acceleration, elastic members are designed for forces determined directly from the global structural analysis or "overstrength" forces based on plastic hinging of the ductile members.

Steel bent caps are required to transfer lateral seismic forces (both longitudinal and transverse) directly through the connections between the bent cap and the superstructure. This can result in significant transverse bending stresses in the bent cap caused by longitudinal seismic forces. In addition, stacked bent caps have an added complexity in that the bent cap may also experience significant torsional forces caused by longitudinal seismic forces.

One way to reduce the seismic force demand on steel bent caps is to detail the steel bent cap bearings to allow longitudinal movement (using guided sliding bearings). This applies to stacked and integral bent caps. On a stacked bent cap, the bearings between the superstructure girders and the bent cap or the bearings between the bent cap and the substructure can be detailed to accommodate longitudinal movement. Likewise, the bearings between the bent cap and the columns for an integral bent cap can be detailed to accommodate longitudinal movement. In these cases, the bent cap seismic forces are limited to the friction generated between the bearing sliding surfaces. Adequate support length needs to be provided.

3.17—EVALUATION OF EXISTING STRUCTURES

Evaluating existing structures includes inspections, load rating, and review of structural adequacy if significant deficiencies are encountered. This Artcile focuses on the load rating aspect of steel bent caps. In addition to load ratings, existing bent caps are often classified as NSTMs and subject to the associated inspection and rating requirements. The most effective way to avoid these requirements is to avoid nonredundant details in new designs or assess the redundancy of an existing structure and demonstrate redundancy, as discussed in Article 3.8.

The load rating of steel bent caps, whether I- or box-section, follows similar methods used for rating steel I-girders and box, or tub, girders. Load ratings are performed in accordance with the load rating provisions of the *MBE*. As of the writing of this document, the most current edition is the 3rd Edition (2018), and all sections and equations cited below are from the 3rd Edition. Load and Resistance Factor Ratings (LRFR) are determined per the *MBE* Part A and *LRFD Design*. Alternatively, Allowable Stress Ratings (ASR) and Load Factor Ratings (LFRs) are determined per the *MBE* Part B and the AASHTO *Standard Specifications for Highway Bridges* (2002). The focus of this section is on LRFR, but much of the discussion applies to all three methods.

In addition to the previously referenced specifications, Owners often have supplemental or overriding rating requirements. These requirements or modifications may include additional legal or permit vehicles, specific condition/system factors, load factors modifications, resistance limit modifications, Owner-specific limit states, or redundant member classification requirements.

LRFR rating factors are calculated in accordance with *MBE*, which is dependent on capacity, C; dead loads, DC and DW; other applicable permanent loads, P; live loads with impact, $LL+IM$; and load factors, γ, as shown in Equation 3.17-1.

$$RF = \frac{C-(\gamma_{DC})(DC)-(\gamma_{DW})(DW)\pm(\gamma_{P})(P)}{(\gamma_{LL})(LL+IM)} \tag{3.17-1}$$

The capacity, C, is dependent on condition and system factors, ϕ_c and ϕ_s, per *MBE* and factored resistances, ϕR_n, which is generally calculated in accordance with *LRFD Design*. For bent caps, dead loads, DC, include the weight of the bent cap and the superstructure elements, including the girders, deck, and barriers they support. Additionally, future wearing surface and utility loads, DW, are applied as applicable depending on Owner requirements and their presence on the structure at the time of load rating. For new designs, DW loads may be included in rating calculations to accommodate their potential future presence; however, per *MBE*, it is permissible to avoid them in the rating of existing structures if they are currently not present. Other permanent loads for load rating, P, include post-tensioning forces, which rarely, if ever, apply to steel bent caps.

Live loads and numerical value of the impact factor vary depending on the applicable limit state, Owner-specified vehicles (i.e., legal and permit loads), and the presence of pedestrian traffic. Due to their geometry and framing, the effects due to braking and centrifugal force transferred from the superstructure to the substructure through the bent cap often need to be considered as part of the live load force. Additionally, like floor beams and other transverse members, ratings for routine permit vehicles may be performed with the permit vehicle in one lane and the design or legal loads in the remaining adjacent lanes. This requires the engineer to determine the lateral placement of the permit truck lane to maximize the resulting live load forces. Refer to *MBE* Article 6A.4.5.4 for specific details and requirements.

The rating of bent caps is subject to the steel structure requirements of *MBE,* and the factored resistances are calculated according to *LRFD Design*. *MBE* Article 6A.6.4 summarizes that the applicable rating limit states for steel superstructure elements include Strength I, Strength II, Service II and Fatigue. Strength I limit state ratings are performed at the Inventory and Operating levels for the design load (typically the HL-93 Design Load) as a screening process for bridges that should be load rated for legal loads. Strength I limit states also apply to legal loads including vehicles specified in *MBE* and state legal loads and emergency vehicles. Legal loads are used to assess safe load capacity and the need for posting a structure. Strength II limit state ratings are performed with permit loads to assess the structure's ability to carry these overweight vehicles. The Service II limit state is performed using design and legal loads to assess the control of permanent deflections. Checking Service II permanent deflections under permit loading is optional and at the discretion of the Owner, which may require deflection checks for routine permit vehicles. The Fatigue limit state, which uses the *LRFD Design* fatigue truck, is optional as well, depending on Owner requirements. *MBE* Article 6A.6.4.1 states that components with fatigue details at a Category C level or lower should be rated for infinite fatigue life. If this is not satisfied, the remaining finite fatigue life can be evaluated using the procedures outlined in *MBE*.

The objectives of design and load rating are not always completely aligned, and these differences need to be recognized, especially during the design of new structures. For bent caps, this is most significant when the component is nonredundant. For design following *LRFD Design*, redundancy may be accounted for by increasing the factored loads by a redundancy modifier, η_R, equal to 1.05 or greater. When rating a nonredundant bent cap, the redundancy is accounted for by applying a system factor, φ_s, per *MBE* to the resistance (see discussion of system factors in Article 3.7). There is no exact match for a single steel straddle beam in *MBE*, as shown in

Table 3.7.2-1. As a result, engineers should consider and check ratings during the design of new bent caps, especially those that are nonredundant. *MBE* states that system factors should be consistent with *LRFD Design* load modifiers but requires those system factors for legal load ratings of nonredundant superstructures.

Triple I-sections may be grouped as a bent cap to provide a redundant member, with the remaining I-sections designed to resist the loads with the loss of one I-section under the redundancy load combinations. The in-service load ratings will typically not control the bridge load rating because these load path redundant caps are designed to withstand the loss of a member.

Steel bent caps are usually I-girder sections, composite tub sections, or non-composite closed rectangular box sections. *MBE* Article 6A.6.9 covers I-girders, including what Articles of *LRFD Design* are applicable. *MBE* states that the cross-section proportion limit provisions of *LRFD Design* need not be considered for rating existing structures. While ratings do not need to be computed based on these proportion limits, the engineer needs to determine if the resistance equations in *LRFD Design* and the research used to develop them are still applicable if these limits are violated. *MBE* also covers box sections in flexure. Similar to I-sections, *MBE* Article 6A.6.11 states that the *LRFD Design* requirements for proportion limits need not be considered; however, when not met, the engineer needs to determine if the resistance equations are still applicable. For box sections, *LRFD Design* covers composite box-section flexure members and non-composite rectangular box-section members. Significant changes and additions were made in the 9th Edition of *LRFD Design* which are not reflected in the 3rd Edition of the *MBE*. The general requirements for box girders in flexure discussed in *MBE*, which correlates to *LRFD Design* provisions for composite box section flexural members, should be used as a guide for the applicability of *LRFD Design* for non-composite sections. For composite I-section and box section bent caps, *LRFD Design* specifies the effective deck width for section properties. *MBE* Article 6A.6.10 covers shear for I- and box-girders by directing the reader to *LRFD Design*. *LRFD Design* also covers shear for composite and non-composite boxes and points back to the *LRFD Design* shear resistance provisions for I-sections.

For the load rating of existing structures, engineers should review the inspection reports to determine if documented section loss or signs of distress should be accounted for in the rating analysis and resistance calculations. Section loss is usually accounted for by locally reducing the gross area of flanges or webs with deterioration. If localized perforations exist in the structural element, consideration should be given to performing or modifying net section checks. Generally, deterioration of steel members is concentrated enough that the effects are only considered in resistance calculations; the minimal effect on the overall structural stiffness does not usually warrant adjusting member section properties in analysis models.

Due to the current relationship of material to labor costs, new designs of steel bent caps generally use thicker webs and flange plates to avoid the fabrication costs, complexity, and fatigue details associated with thin, longitudinally stiffened webs or flanges. However, in older structures where material weight was at a premium in relation to labor, relatively thin plate elements with longitudinal and transverse stiffening were used. At the time of their design, specifications did not prescribe detailed methods or equations for determining the capacity of these slender elements; the effects of local plate buckling, or web bend-buckling may not have been fully considered. As a result, rating these thin plates may produce insufficient ratings with current design specifications. For composite box-shaped bent caps, *LRFD Design* provides equations for determining the resistance of stiffened box flanges in compression. With these equations comes requirements for the minimum strength and stiffness of stiffeners in *LRFD Design*. This article provides reasonable results up to two stiffeners, but minimum requirements past that quickly escalate. Often stiffeners in older existing structures will not meet these requirements.

The effective compression resistance of stiffened noncomposite rectangular box-shaped bent caps is covered in *LRFD Design*. These provisions are based on newer research and can more accurately predict the compression resistance of thinner plates with more stiffeners and provide more reasonable minimum dimensions for stiffeners. If the requirements of *LRFD Design* are too prohibitive for an existing composite box-shaped bent cap, the potential exists to adapt the compression flange capacities of *LRFD Design* noncomposite provisions with Engineer and Owner agreement. If the stiffeners do not meet the *b*/*t* requirements of *LRFD Design*, the commentary provides suggestions for accommodating this with additional information in the FHWA report *Proposed LRFD Specifications for Noncomposite Steel Box Sections* (White, Lokhande, et al., 2019). Additionally, if longitudinally stiffened webs in existing bent caps do not meet the requirements of *LRFD Design* for web bend-buckling and treating the web as unstiffened does not result in adequate capacity, localized 3D finite element models of the web panel can be used to determine a more refined assessment of the flexural compression capacity of the panel.

Force effects, such as shear and major-axis bending, should be considered for rating a bent cap, depending on its geometry and boundary conditions. For a bent cap that supports girders resting on bearings supported on top of the bent cap and acting through its shear center, consideration of vertical shear and major-axis bending dead and live load forces may be sufficient. In these cases, the bent cap forces may be calculated by hand or simple line girder analysis by applying girder reactions to the bent cap at bearing locations. Different combinations of concurrent girder live load reactions should be considered to envelop the results. Using maximum nonconcurrent reactions will usually result in an artificially low (conservative) rating factor. When rating the supported girders, the engineer should consider if the vertical flexibility of the bent cap is significant enough to affect the distribution of negative and positive moments in the superstructure girders or other longitudinal elements (see system stiffness discussion in Article 3.2).

If superstructure girders are framed into the bent cap, major-axis bending moments in the girders induce torsion in the bent cap in the form of opposing lateral flange bending moments for I-section bent caps. If the superstructure girders are curved or skewed to the bent or transmit longitudinal superstructure loads through the bent to the foundation, weak-axis or lateral flange bending moments will develop in the bent cap. Additionally, axial dead and/or live loads may develop in a bent cap depending upon bearing conditions and the orientation of the superstructure to the bent cap. The engineer should determine if any of these effects are applicable and significant enough to warrant consideration. For these cases, a 3D finite element or grid model (including consideration of substructure stiffness) is required to accurately determine the forces and get concurrent results, especially if refined ratings are required to obtain acceptable results. For new designs, these models typically already exist. For ratings of existing structures, a refined analysis model may need to be developed. In these cases, the model results should also be used for the superstructure girder ratings to account for the vertical flexibility and torsional restraint of the bent cap.

Bent caps subjected to combined forces (moments, shears, torsion, and axial loads) should be rated appropriately, considering force interaction where necessary. Torsion effects on I-girders need to be included as lateral flange bending moments. Torsion in box girders (and flexural shear due to lateral moments) can reduce the flexural resistance of flanges as covered in *LRFD Design*. If measurable levels of axial load are present or biaxial bending exists (in box sections), moment-axial interaction should be considered per *LRFD Design*. For these conditions, the resistance is a combination of axial and bending. *MBE* Article A6.10 provides an example of computing rating factors for steel members for the combination of compression plus bending.

SECTION 4—PREFERRED DETAILS

4.1—GENERAL CONFIGURATION

The key to economical bent cap designs is simplicity. Complex detailing leads to increased fabrication costs and potentially expensive maintenance and inspection in the future. The preferred method of detailing will depend upon the chosen longitudinal girder framing options. The three most common longitudinal framing options in use, integral, stacked, and corbel (defined in Article 2.3), are discussed in more detail herein.

4.1.1—Stacked Bent Cap Configurations

As discussed in Article 2.3.2, stacked bent caps are more economical than integral or corbel (non-integral, in-line) bent caps. The detailing of stacked bent caps is generally not complex since there is no need to connect the flanges and webs of the longitudinal girders to the bent cap. Stacked bent caps require more bearings (both between the bent cap and the superstructure and between the bent cap and the substructure); however, the overall cost of the bent cap is more economical in most cases due to simplified detailing.

4.1.2—Integral or Corbel Bent Cap Configurations

As discussed in Articles 2.3.1 and 2.3.3, when there is not sufficient vertical clearance to detail a non-integral, stacked bent cap, an integral or corbel (in-line, non-integral) bent cap may be called for. The geometry of the superstructure often requires that the longitudinal girders are sloped in the longitudinal direction for the roadway grade and that adjacent girders are at different elevations in the transverse direction to accommodate the roadway cross slope. Refer to Article 3.4 for guidance on matching bent caps to superstructure cross-slopes. These geometric requirements need to be accounted for in the bent cap detailing.

4.2—BOX SECTION DETAILING

The typical box girder fabrication sequence is to attach internal diaphragms to the first of the two box girder flanges, attach the webs individually to that flange and then add the second flange of the box girder. Preferred details for welded attachment methods between the webs and flanges are covered by Article 3.2 of AASHTO/NSBA G12.1.

Cost, schedule, and access during the fabrication sequence drive the specified attachment preferences. The type of welding required and subsequent nondestructive testing affects the labor and costs, and therefore the scheduling through the fabrication shop. Even though shop bolting can be common, it is generally not preferred, as it is more labor-intensive and slower to fabricate. However, a bolted connection may be preferred if it enables a bent cap to be classified as an IRM, as discussed in Article 3.7.1. It is essential to consider the size of the box and accessibility when specifying the corner joint details. Most often, one-sided weld joints completed from the outside of the box girder are preferred for the last flange plate added to and enclosing the box girder.

If complete joint penetration (CJP) welds are required for the box fabrication, the design drawings should specify the weld as a CJP rather than a specified joint detail. In doing so, the Fabricator has the flexibility to select the CJP joints that are best for the fabrication sequence, and that can minimize welding or back-gouging within the box girder. Similar to other welded joints in steel bridges, specific weld joint details for the box girders should not be dictated on the contract plans. Other than required weld sizes, plate preparations or joint configurations should not be specified on plans. This allows Fabricators to select weld joints that satisfy the design requirements and suit their expertise, operations, and previously accepted welding procedure specifications, thus providing the most cost-effective solution.

Extension of the flanges past the outer fascia of the web is needed to allow sufficient landing for the web-to-flange weld. Shop and erection practices generally use cradles or lifting rings to carry the boxes, so additional flange reveal beyond what is required for the weld is unnecessary for lifting the girders. AASHTO/NSBA G12.1 has further guidance.

4.2.1—Internal Diaphragms for Box-Section Bent Caps

Internal diaphragms should be welded to three sides of internal box surfaces with a tight fit at the tension flange. Use fillet welds as advised in AASHTO/NSBA G12.1, and avoid complete penetration groove welding because of added costs, added inspection requirements, and potential weld-induced distortion.

Designers should provide an opening for inspection and construction access, as described in Article 3.13. J-clips are recommended over 45-degree corner clips. The recommended size for J-clip/corner clip is 1½ inches × 3 inches for internal diaphragms, as shown in Figure 4.2.1-1. The dimensions of the corner clips should take into consideration the presence of backup bars for complete joint penetration welds.

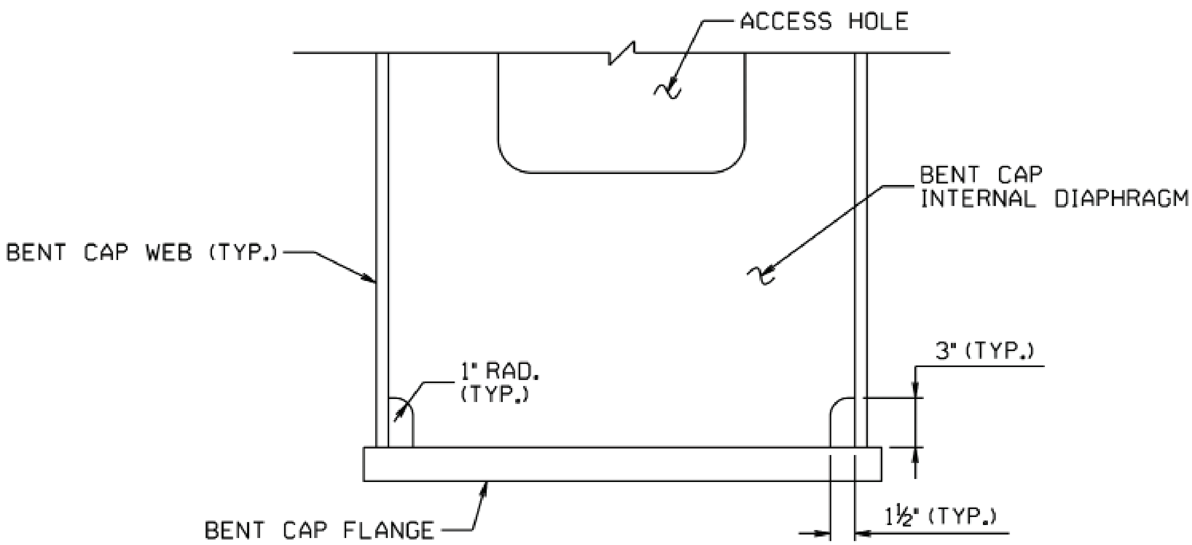


Figure 4.2.1-1—J-clip at internal diaphragm

4.3—TRIPLE I-SECTION STEEL BENT CAP DETAILING

Figure 4.3-1 shows a cross-section of the triple I-section bent cap concept. The connections between the I-sections are bolted plate diaphragms. The advantage of this steel bent cap style is not only redundancy, but also that the bent caps are easier to fabricate than box-shaped bent caps, making them faster to fabricate and more cost-effective. Although the triple I-girder cross-section has advantages, some drawbacks exist. Triple I-girders are also harder to detail for integral bent caps. Compared to a box-section, the narrow work area between the girders in a triple I-girder cross-section means holes for the diaphragms and connection plates should be fully drilled prior to girder fabrication, assembly, and fit-up. For the triple I-section, more components require visual inspection compared to the single I-section or box section. Also, the lateral stiffness of the large girders makes fit-up more difficult than typical I-shaped girders that have smaller flanges.

Plates covering the openings between triple I-section top flanges are bolted to seal the section against water and debris intrusion, while the spaces between bottom flanges are left open to promote air circulation and drainage of any water that happens to condense inside the section. The top flange sealing plates may be included in the computation of the member section properties if the plates are continuous, but it is conservative to ignore them when computing section properties. Open bar grating is placed on top of the bottom flanges to provide a walking surface for inspectors and prevent pest intrusion.

The triple I-section takes more work to fabricate than the twin I-section cap, but it is still less complicated to fabricate than a box section. Both the twin I-section and the triple I-section can be entirely fabricated in the shop and lifted as one member in the field, or the individual I-sections can be erected separately and then bolted in the field (triple I-sections can also be fabricated with two I-sections joined in the shop, with the third section field-installed).

All bent cap sections should provide access for inspection, whether the section is a box or comprises multiple I-sections. Vertical diaphragm plates such as those shown in Figure 4.3-1 have large openings to permit access by fabricators and inspectors. The interior surfaces are typically painted white or another light color so that

Inspectors can more easily see the internal configuration and any defects. Refer to Articles 3.3 and 4.8 for further details.

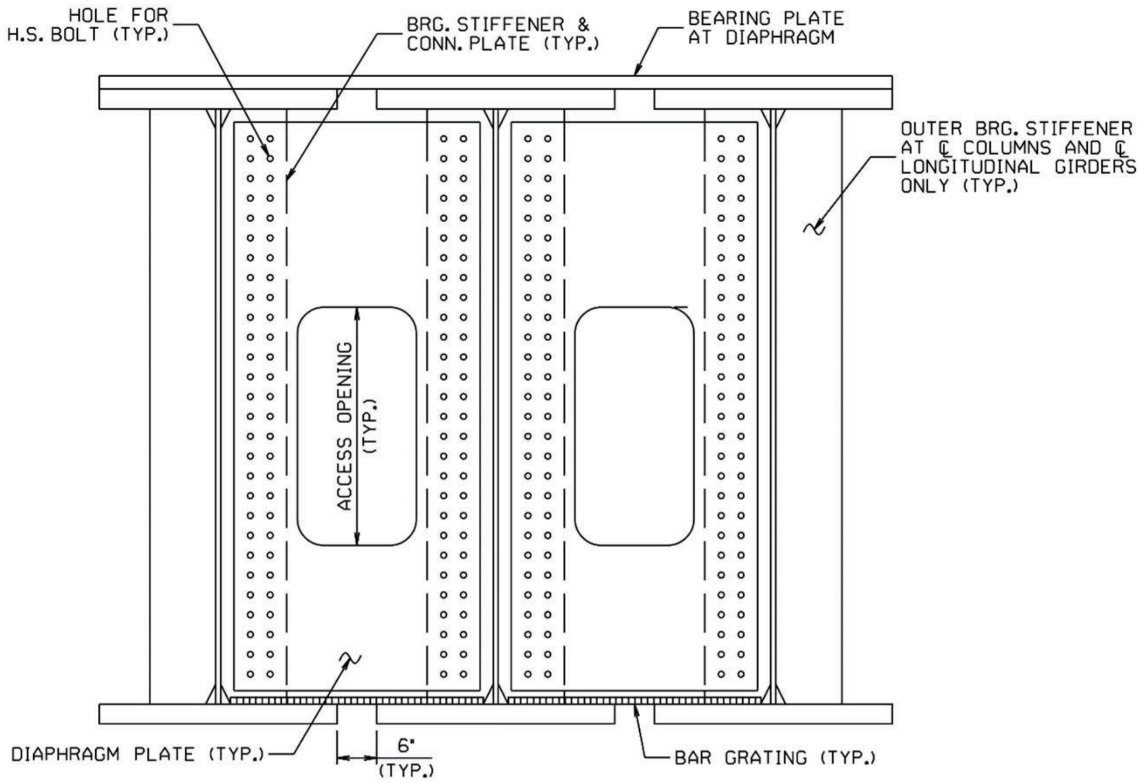


Figure 4.3-1—Triple I-section bent cap

4.4—PREFERRED DETAILS FOR FIELD SPLICES OF BENT CAP

Design of bent cap field splices is addressed in Article 3.14. It can be more challenging to achieve fit-up in box-shaped bent cap field splice connections (which have four sides that need to match) than I-shaped bent cap field splice connections, increasing the importance of taking reasonable steps to facilitate fit-up in the field. Field splice connections will most likely be fabricated with the segments assembled (i.e., match-drilled with all elements in place). If the connections are not drilled in assembly, then the segments should be assembled after drilling to ensure the connection will fit.

Use of oversize holes in bolted field splices is not permitted by *LRFD Design*; however, use of 1-inch-diameter bolts may be helpful compared to ⅞-inch-diameter bolts. Design calculations may allow fewer bolts to be specified in a bolted connection if 1-inch-diameter bolts are used instead of ⅞-inch-diameter bolts. A reduced number of bolts may reduce erection costs. Additionally, the required hole size for 1-inch and larger diameter bolts is ⅛ inch (instead of $^1/_{16}$ inch) larger than the bolt diameter, which may provide a small but useful increase in tolerance. Bolt diameter tolerances increase along with bolt diameter due to manufacturing practices, which means the increased hole clearance of a 1-inch bolt compared to a ⅞-inch bolt will be somewhat less than $^1/_{16}$ inch.

Providing edge distances larger than the *LRFD Design* minimum values is recommended, as prescribed in the “Standard Bolted Field Splices” drawing in AASHTO/NSBA G1.4.

It may be appropriate to design field splices in triple I-section bent caps to reduce shipping lengths or weights. However, recognize that by their nature triple I-section bent caps offer more options to the contractor for shipping, assembly, and erection because the girders can potentially be connected to each other in the field (after shipping to the site), so field splices should be specified as “optional,” as recommended in Article 3.14.

4.5—LONGITUDINAL GIRDER CONNECTIONS

Framing longitudinal girders to integral bent caps is more complicated than stacked bent caps, as the connection of both flanges presents numerous fabrication and erection issues. Due to the complexity of the connection details, Fabricators typically shop-assemble the longitudinal girders and integral bent caps connections in order to verify fit-up, adding considerable time and effort to production; such shop-assembly is typically not necessary for stacked bent caps.

At the connections of the girder flanges to the caps, two-way beveled fill plates are sometimes needed to reconcile the geometry between the longitudinal girders and the bent cap. As mentioned in Articles 2.1 and 3.6, for detailing and design it is preferable to orient longitudinal girders perpendicular to the bent caps. This is true for both integral and non-integral bent caps, as a skewed condition complicates the geometry and often leads to the need for two-way beveled fill plates. Furthermore, it is preferable to avoid cross-slope transitions within the longitudinal girders where they frame to the bent caps, as this can also trigger the need for beveled sole plates or variable-height bolsters. If possible, configure the geometry to avoid the need for two-way beveled fill plates as they are costly to fabricate and complicate erection. If two-way beveled fill plates are needed, ¾-inch minimum beveled plate thickness is preferred.

The detailing of I-shaped longitudinal girder top flanges above integral bent caps is a suitable example of where two-way beveled fill plates can be avoided. Longitudinal girder continuity can be achieved by bolting the longitudinal girder webs and bottom flanges to the steel bent cap while extending the girder top flange tension tie plate across the top of the steel bent cap. The tie plate is bolted to the plate to the top flanges of the trailing and forward longitudinal girders but floating above the bent cap, leaving an air space between the bottom of the tension tie plate and the top flange of the bent cap (eliminating the optional two-way beveled fill plate shown in Figure 4.5-1 and Figure 4.5-2). This space should be a minimum of 3 inches high to avoid debris accumulation and associated corrosion. The web connection is designed to resist shear only. The compression in the longitudinal girder bottom flanges is accommodated by bolting T-shaped weldments to the bent cap and then bolting the longitudinal girders to the T-shaped weldments, as shown in Figure 4.5-1 and Figure 4.5-2. The long legs of the weldments have holes that match holes in the bottom flanges of the longitudinal girders. The weldment and the bottom flanges are bolted together, resisting the compressive force as shear in these bolts. An alternative method of resolving the compressive force in the bottom flanges of the longitudinal girders is to machine to bear the end face of the longitudinal girder bottom flanges and the outside faces of the steel cap girder bottom flanges. However, this is not recommended as machining these surfaces to bear is difficult, time-consuming, and costly. Detail the web to avoid distortion-induced fatigue, considering the elevation difference between the bent cap and longitudinal girder bottom flanges. If necessary, a flange continuity plate (horizontal stiffener) that transfers the compressive force from one web of the box to another should be installed inside the box at the elevation of the longitudinal girder bottom flange, as shown in Figure 4.5-1 and Figure 4.5-2. Provide sufficient welder access for the flange continuity plate and corner or J-clips as shown in Figure 4.2.1-1.

Erection considerations should also be addressed when framing longitudinal girders to integral bent caps. The longitudinal girders are shop assembled with the bent caps in a fully cambered position. This position, or a portion of it, should be field imposed for the longitudinal and transverse framing to fit. This can generally be imposed with a crane, but may require falsework.

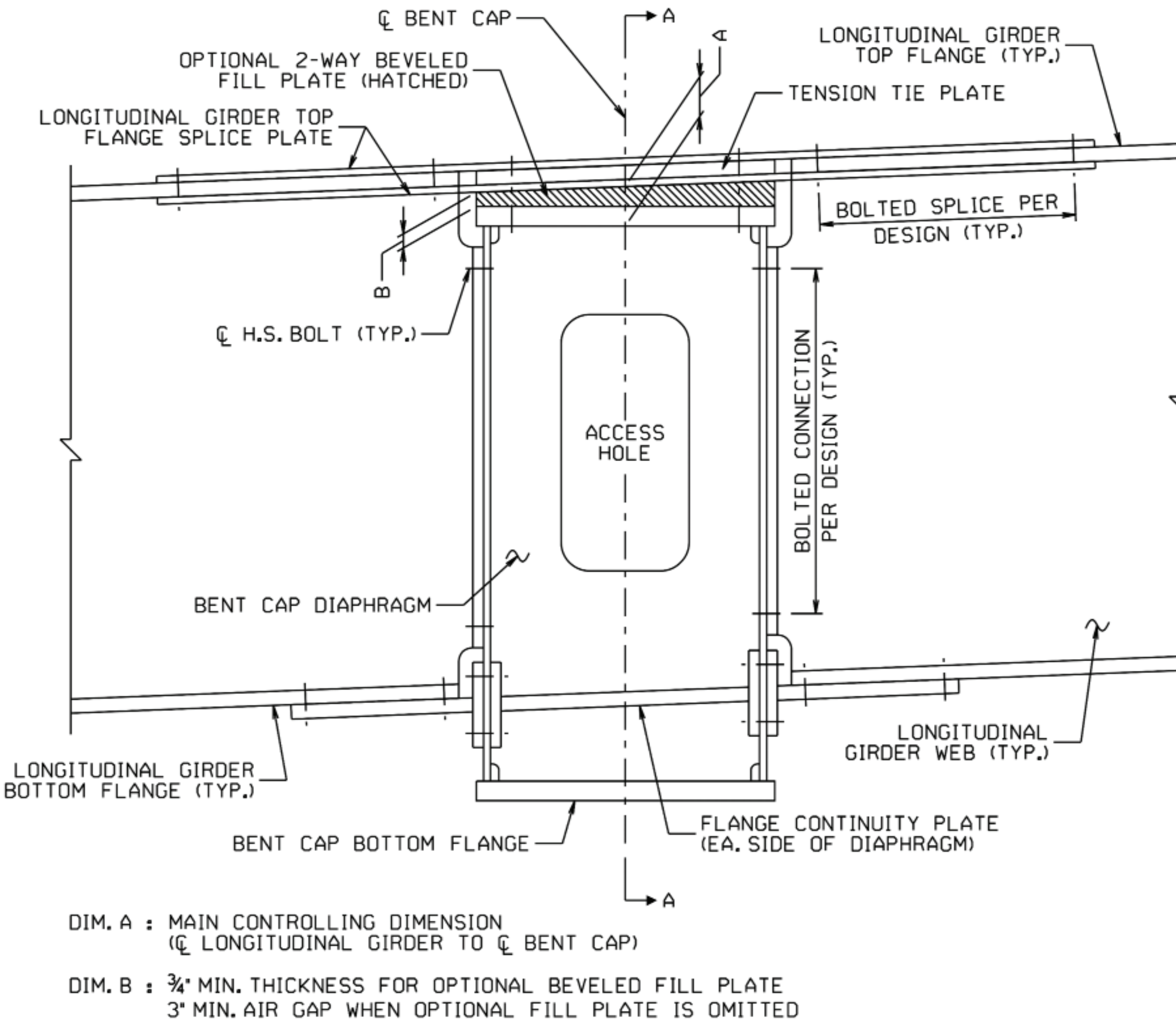


Figure 4.5-1—Longitudinal girder connection to steel box bent cap

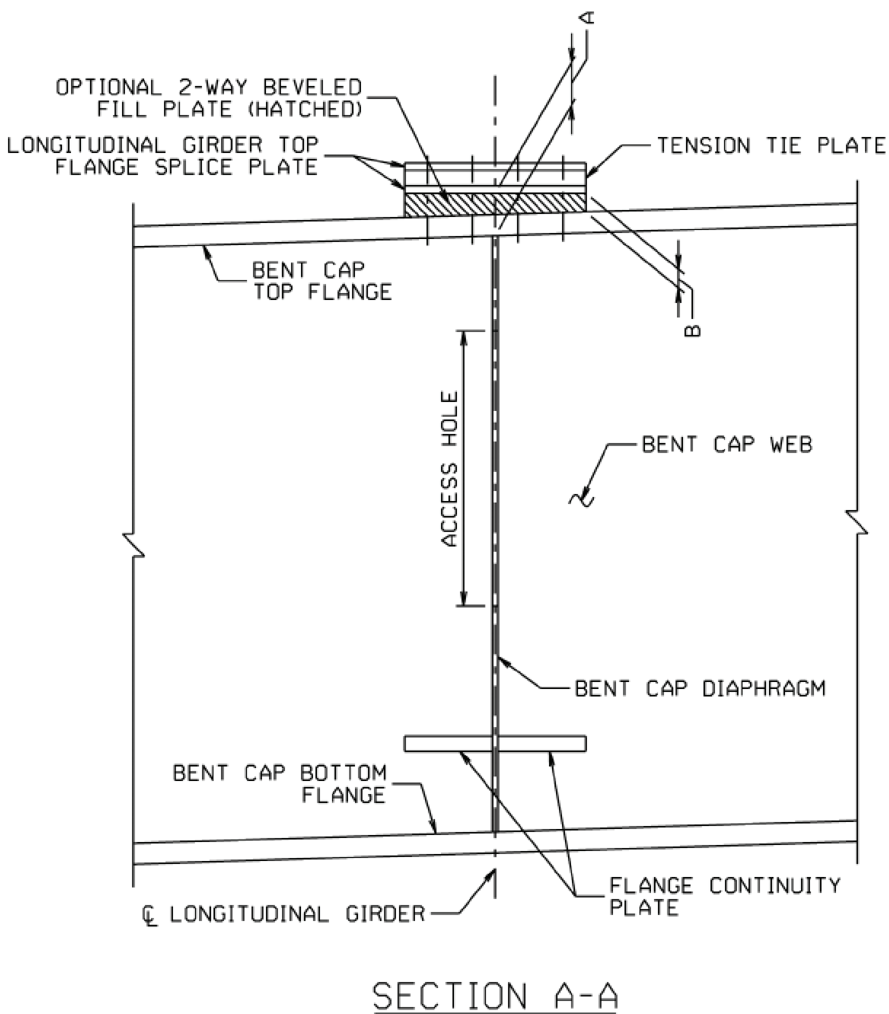


Figure 4.5-2—Longitudinal girder connection to steel box bent cap, cross-section

4.6—SAFETY

The primary safety considerations and concerns of steel bent caps are driven by the size and weight of the members and the enclosed spaces created by box girder sections. The bent cap members are typically large, heavy components, and thus, handling and rolling these members requires careful planning and execution in the fabrication shop. Generally, the weight of individual bent cap shipping pieces should be limited to 75 tons and they should be of sufficient length for axle load distribution (e.g., for a 75-ton piece, 70 feet of length will generally facilitate shipping). While some Fabricators can handle pieces over 100 tons, and the actual geometry and configuration of the piece affects specific handling limits, 75 tons can be handled and rolled safely and efficiently by most Fabricators involved in these types of bridge projects. Heavier pieces can dramatically increase fabrication costs as unique fixtures may need to be designed and manufactured by the Fabricator.

There are increased safety concerns for box-shaped sections compared to I-shaped sections. Consideration should be given to the fabrication and construction tasks required inside box members. Adequate access for any operations—welding, weld testing, bolting, painting, inspection, etc.—should be provided. Entry points into the box girder and access holes through internal diaphragms within the box girder should be as large as practical. The sound, sparks, and heat generated from welding box shapes create significant safety concerns that the fabricator should consider. For the fabrication of the third and fourth side of a welded box section bent cap, consideration should be given to being able to install the weld from the outside of the box; backing bars help to permit fabrication from the box exterior. Refer to Article 4.2 for further detail.

Enclosed spaces create the most significant safety concern for fabricating closed box sections. Often these spaces are defined as confined spaces by OSHA. Confined spaces will be considered either permit-required or non-permit-required confined spaces. Generally, permit-required confined spaces have the potential to contain some hazard (e.g., atmospheric, physical, engulfment). During the fabrication process, welding fumes and other process by-products have the potential to become an atmospheric hazard. Thus, box sections are considered permit-required confined spaces. During the coating process, the main concern is flammable atmospheres encountered due to vapors from the coating products. Under the permit-required confined space entry standard 29 CFR 1910.146, all employees who will be entering a confined space are required to be trained on the hazards of the space and procedures and methods for controlling those hazards.

4.7—BEARING DETAILING

For non-integral steel bent caps, the bearings above the bent cap that support the girders are generally HLMR or elastomeric bearings. The connection details will vary depending on the bearing selected, and selection guidance is available in AASHTO/NSBA G9.1, *Steel Bridge Bearing Guidelines*. HLMR bearings are typically designed and fabricated with both a base plate and masonry plate below and a load plate and sole plate above. For bearings supported by steel bent caps, the masonry plate rests on the top flange of the steel bent cap and not on a masonry support. For bearings having masonry plates, for HLMR bearings, and for any bearing line requiring fixity, the bearings for the longitudinal girders should be connected to the top flange of the steel bent cap. These plates can be bolted or welded to either the girders or the bent cap in any combination. When a Designer chooses a welded connection for the bearings above the steel bent cap, consideration of the fatigue design of the cap is critical.

If the straddle bent is a simple span, the bearings can be welded to the top flange, as it is always in compression. However, if the straddle bent is continuous over one or more supports, or if the steel bent cap is in a hammerhead configuration, some or all of the top flange is in tension; load-induced fatigue evaluations are required and could prompt the use of bolted bearing connections to the steel bent cap. Welds transverse to the bent cap axis produce Category C fatigue details, and welds longitudinal to the bent cap axis may be Category D, Category E, or Category E′ welds, depending upon the length of the weld and the thickness of the attachment at its termination. Refer to Section 7 of Table 6.6.1.2.3-1 in *LRFD Design* for categorization. In these cases, the masonry plate can be bolted to the top flange to eliminate welded fatigue detail connections to the top flanges. In addition, masonry plates often need to be beveled in at least one direction to take transverse roadway cross slope into account.

If elastomeric bearings are designed to support the girders on the steel bent cap, the designer needs to consider the attachment of the elastomer to the bent cap. At concrete substructures, many states allow the elastomeric pad to bear directly on the concrete and take advantage of friction to limit the movement of the bearing when lateral displacement is induced into the bearing. On a steel bent cap, friction is significantly reduced compared to a concrete bent cap, and designer should investigate whether restraint is required for the elastomeric pads to prevent the elastomeric pads from "walking" when lateral displacements occur. If necessary, the bearing pads can be positively attached to the bent cap using a vulcanized base plate (masonry plate) bolted or welded to the cap or by using welded or bolted keeper bars surrounding the pad. The elastomeric pad may also require vulcanization to a sole or connection plate for attachment to the girders. Further information and details are available in AASHTO/NSBA G9.1.

If corbels are used, the horizontal area of the corbels should be large enough to accommodate the largest horizontal dimensions of the bearings that they support. Generally HLMR bearings are smaller and fit more easily than elastomeric bearings. The general issues noted for bearings on top of bent caps also apply to bearings on corbels. Some existing bent caps have corbels framed into the web of the bent caps. The longitudinal girders are supported on bearings which are supported by the corbels.

For future maintenance and replacement, the Designer should provide means for bearing replacement in the design and detailing of steel bent caps. This includes the bearings which support the steel bent cap and the longitudinal girders supported by the steel bent cap. In the steel bent cap and column designs, the engineer should include jacking forces for bearing replacement in the design, including full dead and live loads. If jacking stiffeners are provided in the steel bent cap, they may require close spacing to other elements. Access between stiffeners should be sufficient for fabrication and inspection. The design should also provide sufficient space to accommodate the jacks (including both sufficient vertical clearance and a large and strong enough horizontal surface). In addition, pockets prone to collecting debris and corrosion should be avoided. Bearings may include separate steel load plates or other elements welded to the sole plate and/or masonry plate. This allows bearing replacement to be performed without removing the sole plate and/or masonry plate, which could result in reduced bearing fabrication costs or eliminate demolition or partial demolition of the concrete columns for bearing replacement. Bolted connections make this very easy but may require an unrealistic number of bolts depending on the bearing forces. Bearing components can be recessed into the base and load plates to resist lateral forces and still provide a future replacement when the plates need to be welded.

Variations in bearing seat elevations will require additional detailing and consideration. Bearing pedestals and fill plates can fill the difference in elevation between the bearing seat and the top of the bent cap and will be required for a level bearing surface if the steel bent cap is sloped. Pedestals or bolsters are preferred for large differences in bearing seats, whereas fill plates or varying sole/base plate thicknesses can be used for bridges with shallower differences. Pedestals can be constructed using plate material to form a box on four sides with a level bearing plate on top and vertical stiffeners as needed (see Figure 4.7-1).

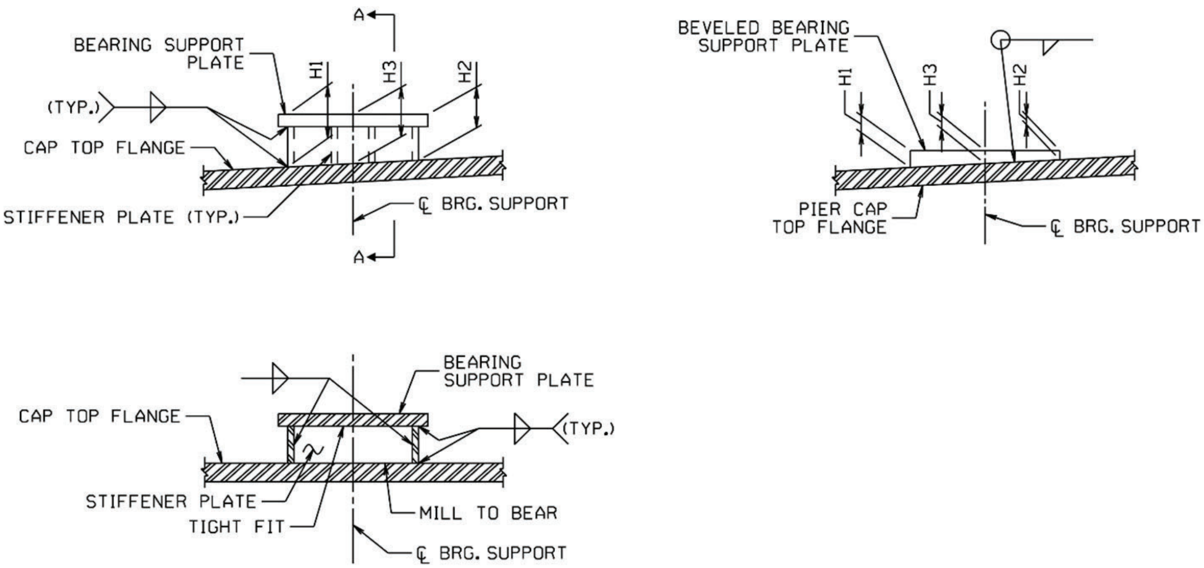


Figure 4.7-1—Bearing support plates

Attachment of the bearings below the bent cap to the supporting column is typically accomplished using anchor rods embedded in the concrete columns and connecting to a masonry plate. If feasible, use a bearing system that is forgiving of anchor rod mislocation at the expansion ends of the bridge. The bearing can then be bolted or welded to the bent cap. For simple span bent caps, the welded attachment has a negligible impact on the fatigue design because there is minimal moment at the ends of the cap. For long bent caps which are continuous over an intermediate column, the bottom flange is in compression and so welding the flange to the bearing pad load or sole plate is a suitable option.

The bearing load between the steel bent cap and supporting column may consist of the downward portion of a force couple due to unbalanced loading or the uniform vertical compressive load from balanced downward forces over the full width of the top of the concrete column. To distribute that load, a thick steel bearing (masonry) plate that is wider than the steel bent cap can be placed between the top of the concrete column and the bottom flange of the bent cap. The masonry plate should be wider than the steel bent cap for two reasons. First, a wider masonry plate will decrease the bearing pressure on the concrete below by increasing the surface area of steel bearing on concrete. A 45-degree angle distribution of load from the narrower bent cap bottom flange to the wider masonry plate is assumed, so an increased width for this reason alone would only require an increase of width equal to twice the masonry plate thickness. The second reason applies in locations where Owners require or prefer the use of anchor rods with masonry plates. In that situation, the masonry plate width needs to increase in order to accommodate the rods. Depending upon rod diameter, this typically requires a masonry plate that is approximately 3 inches wider on each side of the bent cap bottom flange.

4.8—ELECTRICAL AND LIGHTING

Since most bent caps, as discussed in Article 6.1, are required to have a hands-on inspection on a routine basis, Designers should detail bent caps for easy inspection access. For bent caps consisting of a box or multiple I-shaped cross-section, it may be prudent to provide electrical power and lighting fixtures to assist in achieving an efficient and productive inspection. Furthermore, as box sections are enclosed areas, the inside temperature may be substantially higher than ambient temperature. By providing ventilation and electrical power or provisions for a portable generator at each end of the box girder, inspectors can use fans to circulate air and lower the temperature while they are inside the box.

If the Owner decides an electrical system and lighting is required, contract documents should include interior lighting fixtures and electrical outlets at locations such as access openings and at spacing not exceeding 50 ft along the length of the box. Additionally, locate lighting switches at entry access openings for the convenience of the inspectors. Providing adequate lighting for the arms-length inspection is encouraged. A qualified engineer should specify the design and materials for electrical wiring/conduits and lighting fixtures. The actual layout of conduits is detailed through the shop drawing process; however, the engineer should specify any requirements regarding attachment methods or attachment locations of the conduit. A few further recommendations, if a lighting system is provided, are:

- Provide six-hour reset timers for each circuit to turn off the lighting system automatically,
- Use wire guards on light fixtures,
- Use 316 stainless steel for supporting hardware, and
- Paint steel bent cap interior as shown in Figure 4.8-1 and discussed in Article 3.3.

Figure 4.8-1—Interior of steel box-shaped bent cap

4.9—DRAINAGE AND VENTILATION FOR CLOSED STEEL BENT CAPS

Box girders can be designed to be sealed against moisture infiltration or to be well-drained. Coordinate all aspects of box design toward one or the other of these designs. However, if seals deteriorate, water can enter a closed box girder or water may condense inside the box-shaped section and so drain holes are recommended. If drainage holes are provided, design each box girder with minimum 2-inch-diameter ventilation or drain holes located in the bottom flange on both sides of the box spaced at approximately 50 feet or as needed to provide proper drainage. Place drains at all low points against internal barriers. Install screens or vermin guards to keep birds and bats out of the boxes as they can plug the drains. Require a 0.25-inch mesh screen on all exterior openings not covered by a door. Welding of the screen to structural steel components should be avoided. Instead, the screen should be attached to structural steel components with adhesive. This includes holes in webs that pass utility pipes, ventilation holes, drain holes, etc. Refer to AASHTO/NSBA G1.4 for further guidance.

SECTION 5—FABRICATION AND ERECTION

5.1—SHOP ASSEMBLY

When sufficient vertical clearance does not exist for a stacked bent cap, either an integral or non-integral in-line bent cap is typically used, as shown in Figure 5.1-1. Consequently, the framing system becomes more complex, and when integral, the system is more rigid in the vicinity of the pier. For such integral configurations, alignment is usually verified via shop assembly of the bent cap, including field splices to adjacent longitudinal girders, for a minimum of one full field section beyond the first splice. As shown in Figure 5.1-1(c), the stub projections are considered integral with the box, so longitudinal girder field sections should be assembled to check alignment relative to tolerances. Suggested values are listed in Article 5.2.

(a)

(b)

(c)

Figure 5.1-1—Bent caps, with (a) in-line, integral bent cap, (b) stacked bent cap, and (c) integral cap in shop assembly

5.2—TOLERANCES

The *AASHTO LRFD Steel Bridge Fabrication Specifications* (AASHTO, 2023a) specify dimensional (length, depth, flange tilt, etc.), horizontal sweep, and vertical camber tolerances for individual field sections, as well as for beams and girders spliced together longitudinally within bridge spans. The tolerance recommendations listed below are specific to integral bent cap assemblies and their adjacent moment-connected longitudinal girders. Although these tolerances are not included in the *AASHTO LRFD Steel Bridge Fabrication Specifications*, the allowable variation from specified camber listed below is considered reasonable and achievable for the assemblies discussed in Article 5.1.

- For single-span bent caps with longitudinal I-girders or tub girders reamed into moment connections at the bent cap:
  - –⅛ inch, +¾ inch
- For single-span bent caps with longitudinal girders computer numerical control (CNC)-drilled:
  - –⅛ inch, +⅜ inch
- For continuous multi-column, hammerhead, or cantilevered steel bent caps:
  - –⅛ inch, +⅜ inch

For bent caps such as those shown in Figure 5.1-1, it is essential to consider the potential effects of compounding tolerances of the entire assembly so that the aggregate of the longitudinal girder camber combined with the transverse bent cap camber does not create excessive additive positive or negative camber variance. For this reason, the shop assembly of an integral steel bent cap often includes the first longitudinal field sections. Where beams or girders pass over the steel bent cap, conventional camber and sweep tolerances as described above can similarly be cumulative and should be monitored, but complete unit shop assembly is not generally warranted.

Traditional bearing point tolerances for plan and elevation are typically specified by the Owner. In absence of an Owner-specified tolerance, a recommended elevation variation tolerance between adjacent beam seats is listed in AASHTO/NSBA G9.1. Substructure tolerances, survey accuracy, and tolerances of the complex geometry associated with steel bent cap assemblies are cumulative and affect the bearing points of longitudinal girders.

Integral bent caps have special shop alignment considerations as described above. When connections between longitudinal girders and bent caps are reamed or drilled in assembly, the practical limits are larger than if girders are CNC-drilled full-size. For transverse, single-span bent caps with longitudinal girders that are CNC-drilled, the allowable variation from specified camber for the bent cap camber is more limiting to reduce the risk of misalignments occurring at the shop assembly fit-up check (corrected as needed by re-making customized splice plates). In addition, bent cap tolerances are often held more tightly to ensure fit-up of the framed-in connections depending on the various framing configurations.

Figure 5.2-1 illustrates typical plan dimensions that can be measured to check compound tolerance for an in-line, non-integral or integral steel bent cap shop assembly with adjacent field sections. Elevations are checked at bent cap bearing points and adjacent field splices (especially exterior girder locations). Provided the transverse/radial spacing and longitudinal (length) dimensions shown are correct, diagonal, sloped length dimensions falling within ±⅜ inch of theoretical (adjusted for steel temperature) are indicative of an assembly that falls within the plan and elevation aggregate tolerances as suggested above.

These issues may require tighter tolerances for differential girder sweep and camber than usual I-girder tolerances. The Fabricator assembles the box and therefore can establish the camber and sweep tolerances as needed to assemble the cap. Normal fabrication tolerances have proven to be adequate for typical girder bridges; however, minor camber and sweep adjustments may be required to achieve proper fit-up of closely spaced and stiff bent caps.

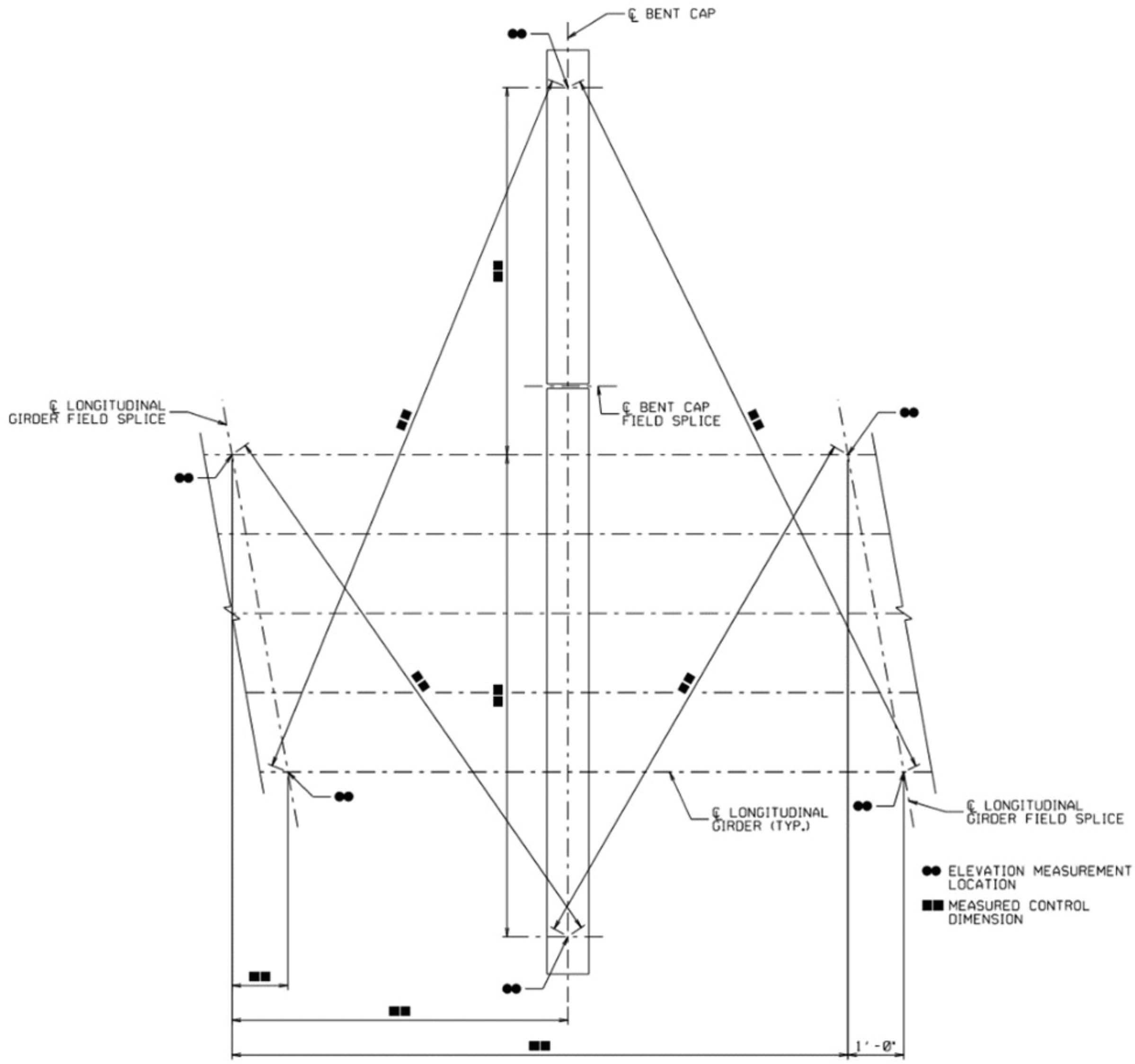
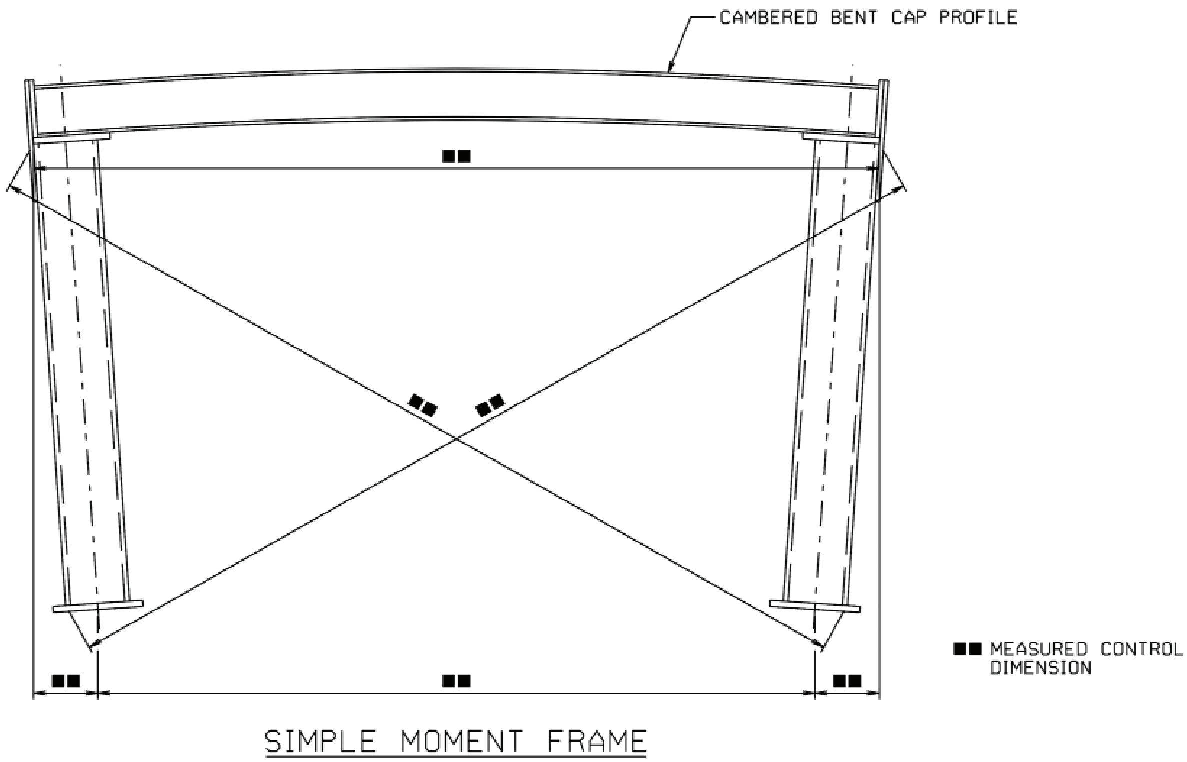


Figure 5.2-1—Typical plan dimensions for checking tolerances

5.3—CAP-TO-COLUMN CONNECTIONS

Traditional transverse bent caps are typically supported by conventional steel laminated elastomeric bearings or HLMR bearings, which are anchored to reinforced concrete columns. The framing of the superstructure girders and bent cap will affect the type and detailing of the bearings used.

Occasionally, the bent columns consist of bolted moment connections in a simple or continuous frame configuration. In such cases, longitudinal girder field sections are assembled with the transverse bent cap to the cambered profile, as shown in Figure 5.3-1. The bent cap is then laid on its side and separately assembled to the columns in a lay-down assembly such as those shown in Figure 5.3-1, usually fully cambered to the no-load profile (note the splay of columns, which are usually not cambered). Cap-to-column connections are then scribed to finished end-cut length in assembly (usually inside the Fabricator's shop, or pre-dawn so that thermal distortion does not occur), adjusting for neutral temperature, then machined to bear against the underside of the bent cap moment connection and re-checked in assembly.



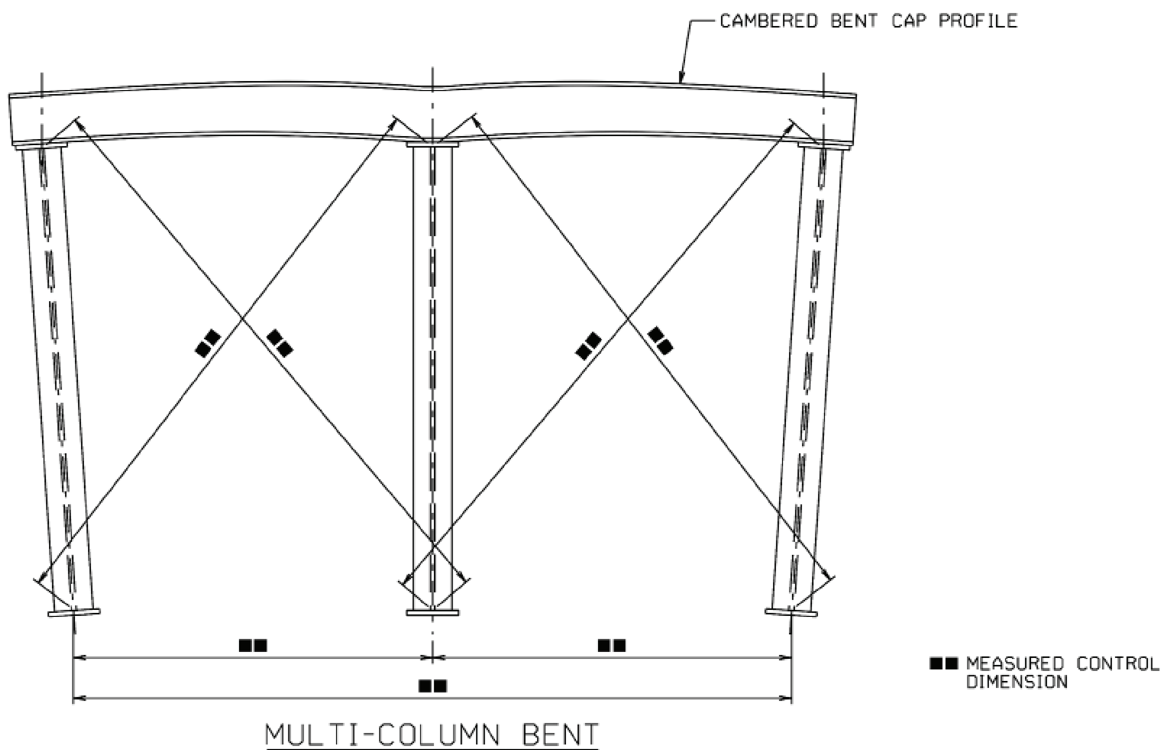


Figure 5.3-1—Straddle bent piers with steel columns at no-load profile lay-down assembly

5.4—ERECTION

The erection of steel bent caps in the field is similar to steel girder erection. AASHTO/NSBA S10.1 provides useful guidance for steel bent cap erection. Steel bent caps in their final condition are often very long and heavy members. As such, the field erection should take into account several considerations to safely set them in place. Occasionally the bent caps are transported to the site in multiple sections spliced together to make the completed assembly. A no-load blocking diagram should be shown on the erection procedure if a field splice is required in the steel bent cap, which is usually spliced on the ground at the site.

Control of geometry is essential for multiple span bridges constructed with integral bent caps. To achieve correct longitudinal geometry, erectors may need to incrementally rotate steel bent caps to maintain the correct geometry of longitudinal girders as construction progresses. Adjustments can be made with multiple cranes, hydraulic jacks, or come-alongs with deadmen. In addition, the erection engineer should ensure that the stability of the steel bent cap is maintained during the placement of the longitudinal girders. A stop may be incorporated into the bearing design to maintain stability during erection. Lateral restraints may be required to avoid excessive lateral deflections during erection. Resistance against rotation can also be achieved temporarily during construction by temporary collars, which lock the bearings in place until the cap is stabilized. This operation requires careful planning and supporting calculations for geometric rotation, jack pressures, etc. Figure 5.4-1 shows rotation of a steel bent cap controlled during construction with the use of wood shims at the bearings and come-alongs. Figure 5.4-2 shows the use of wood shims for bracing bent caps against columns.

Figure 5.4-1—Erection of a steel bent cap

Figure 5.4-2—Geometry control of a steel bent cap

Integral, "diving board" type corbels can serve as erection seats to help land and stabilize the longitudinal girder in place while the splice connection is made. Figure 5.4-3 shows "diving board" type compression flange splice plates with corbel stiffeners. In the figure, the integral bent cap is laterally stabilized by a pair of erected girders connected to the near side web. The cable shown was required until the second girder was erected with sufficient cross-framing and was subsequently removed during the next night's lane closures.

For multi-span, continuous bridges with a series of integral, in-line steel bent caps, it is crucial to be able to control the rotation of the transverse caps as the top flange, web, and bottom flange moment connections are made across sequential spans (or when dropping in a span where adjacent approach spans have been previously erected, as might be required for traffic staging). It may be necessary to jack from back-span towers or lift the longitudinal girders with cranes to facilitate connections of forward (drop-in) spans. In such cases, cross-frame connections should not be tensioned until integral moment connections are completed. Figures 5.4-3 and 5.4-4 employ the methods described. In this case, stay-cables, come-alongs, and bearing collars were used to control the bearings, and back span B may have to be adjusted as erection progresses toward span C.

Figure 5.4-3—Supports for longitudinal girders at integral steel bent cap

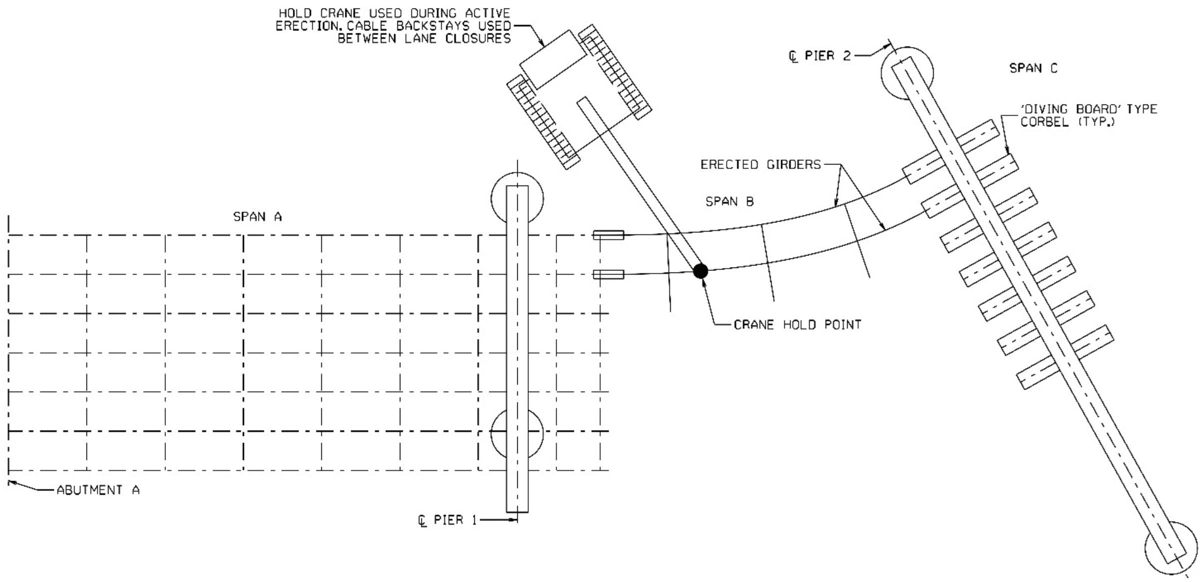


Figure 5.4-4—Schematic erection plan

Figure 5.4-5—Erection of steel bent cap

Another means of erecting continuous spans with large straddle bent caps is shown in Figure 5.4-5. For this bridge, the box girder was rigidly held in place, meaning rotation was restrained about the bent cap longitudinal axis, as balanced cantilever segments of the longitudinal girders were subsequently erected. The bridge shown in Figure 5.4-4 and Figure 5.4-3 was built by a different erector and under a separate contract from the bridge shown in Figure 5.4-5, which illustrates how contractors may employ their individual means and methods toward a common goal.

5.5—ACCELERATED BRIDGE CONSTRUCTION

Bent caps can be used in conjunction with Accelerated Bridge Construction (ABC) methods. The key to successful ABC projects is simplified detailing. Non-integral, stacked bent caps should be the first choice for ABC projects due to the simplified connections between the superstructure and the bent cap. The designer should verify the stability and strength of the bent cap during erection. An example is a stacked bent cap combined with the use of simple-span modular deck beams (MDBs) (beam pairs or triplets combined with a plant-cast concrete deck), optionally made continuous for live load. The two eccentric bearing lines of the MDBs will result in torsional forces on the bent cap.

If an integral bent cap is required, it is still possible to use ABC methods. One option is to detail a longitudinal beam splice near the bent cap (approximately 5 feet on either side of the bent cap). This will allow for the erection of the bent cap with the complex connection details in place, leaving relatively simple beam splice connections to complete the superstructure.

SECTION 6—INSPECTION AND MAINTENANCE

The inspection of steel bent caps of highway/roadway bridges is governed by *MBE* Article 4.2.5 and by the requirements of individual Owners. As steel bent caps are structural steel members with a portion of the member in tension, and most are non-redundant (see Article 3.7), they are commonly considered NSTMs. NSTMs require that the Inspector be arm's length from any portion of the member that is in tension. This requires the use of access equipment. Inspectors can use ladders over the portions of the bent cap span that pass over relief areas, but portions spanning over roadways require bucket trucks with baskets, boom arm trucks with baskets, or self-propelled aerial lifts. All of these options require maintenance and protection of traffic of the roadway that is crossed, including lane closures. An under-bridge inspection unit truck with an articulated arm can sometimes be used to inspect the bent cap, depending upon the length of the bent cap; however, doing so requires lane closures and maintenance of traffic of the roadway that is carried, and when using an under-bridge unit truck there should be sufficient vertical clearance above the roadway that is crossed. During the initial inspection, the general configuration of the bent cap and an example of each of the different types of details should be photographed and placed in the inspection report, so that there is a baseline of initial conditions against which future Inspectors can compare future defects.

For all types of bent caps, inspection of bolted connections and welded details in tension zones are the highest priority for Inspectors. The termination points of vertical connection plate welds, vertical welds for corbel connections, welds connecting bearing base plates to top flanges in tension, and the exterior of corners of welded box bent caps are critical places to look, as cracks may generate from these locations.

For twin I-section, triple I-section, and box section bent caps, the interior areas of the bent cap should be inspected. For all three types of bent caps, this usually requires that the interior space is adequate for a person and that there is an entrance/exit hatch. For twin and triple I-section members, if there is a removable grating spanning between the bottom flanges of the individual I-sections, access may be gained by removing the grating. The Inspectors will need to supply their own lighting if conduits and lighting have not been installed as recommended in Article 4.8. A combination of floodlighting and individual lighting might be needed to see individual details while ensuring that the path to the entrance/exit hatch is always visible. If permanent electrical service is not provided within the box, a portable generator can be used, but must be located so that exhaust fumes from the generator are not blown into the box. The applicability of OSHA confined space requirements (see 29 CFR 1910.146) should be considered.

When inspecting the interior of a box section bent cap or interior portions of twin and triple I-section bent caps, the need to take special care looking at welds in tension zones and any connections also applies to the exterior of the bent cap. The same inspection tools and documentation requirements as for the exterior of the bent cap hold for the interior as well. Additionally, the Inspectors should look at the welds connecting the interior diaphragm plates, particularly at corners in the tension zone, and at the interior corners of the boxes. If there are backing bars that are to remain in place, the welds connecting the backing bars to the flanges and webs are critical locations to be inspected in the tension zones. For modern box girders, the backing bars should have been made continuous for the full length of the box girder. If they have not, the gap between bars should be inspected closely for cracking; if the bars have been made continuous, the welds that make them continuous should be inspected. If there are signs of distress in any welds, the inspection firm and the Owner can decide if it is necessary to conduct additional nondestructive examination of defects, such as ultrasonic testing. The interior of box girders should be inspected for the locations (if any) of internal drainage holes or pipes unless it is a fully sealed box, and all inspections of the interior of box girder bent caps should note the presence of any standing water.

The bearings of the bent cap should also be inspected, both those of the longitudinal girders (if any) and those of the bent cap. If there are elastomeric bearings, it should be checked that the bearing is deflecting in the logical direction based on that day's ambient temperature. The Inspector should measure the amount of elastomeric bearing deflection and record it along with the temperature. If there are welds connecting the sole plate to the bent cap bottom flange, the Inspector should inspect them for cracking, and inspect any welds connecting longitudinal girder bearing base plates to the top flange of the bent cap. If masonry plates of longitudinal girders are connected to the bent cap top flange with bolts, the Inspector should check the condition and tightness of the bolts, check if any of the bolts have fractured, and check if there is any leakage through bolt holes in the top flange of the bent cap. The anchor rods should be sounded with a hammer to ensure they have not fractured. The Inspector should check the presence and the condition of any nuts for the anchor rods. The conditions of the bearings should be documented and placed in the report in the same amount of detail as for the bent cap itself. During an initial inspection, the Inspector should also check that the bearing sizes match the as-built plans, and check that the allowable movement of each bearing is oriented in the same direction or directions as shown in the as-built plans. Also, during an initial inspection, the Inspector should measure the diameter of the bolts connecting longitudinal girder base plates to the bent cap top flange, check the diameter of the anchor rods embedded in the concrete columns, and check the condition of these anchor rods and that they are properly embedded in the concrete.

6.1—IN-SERVICE INSPECTION FOR FATIGUE AND FRACTURE

The classification of a bent cap as an NSTM or IRM has consequence in terms of in-service inspection, and this is where designs with load path redundancy, like the triple I-girder bent cap, and designs with internal redundancy are beneficial. Per the 2022 NBIS, NSTMs require regular "hands-on" inspection every 24 months. The "hands-on" requirement recognizes that NSTMs are susceptible to fatigue cracking, and Inspectors' being up-close to details that are prone to cracking allows for plenty of time to mitigate such cracking before it propagates as a fast fracture. The "hands-on" requirement often leads to increased cost over routine inspections because of the need for up-close inspection access (i.e., maintenance of traffic and need for specialty inspection vehicles). Bridges without NSTMs also have routine inspection intervals of 24 months, but do not require the "hands-on" access. Both NSTM and routine inspections can have extended intervals out to 48 and 72 months, respectively, with a written policy and notification to FHWA.

The 2022 NBIS allows for Owners to reclassify NSTMs as IRMs. However, FHWA requires the Owner to define a written procedure for when this can occur. The written procedure should follow a nationally-recognized method (such as the AASHTO *IRM Guide Specs*) and include analysis requirements, special detailing guidance, routine inspection requirements, special inspection requirements, and evaluation criteria as to when the IRM designation would revert back to NSTM. The last point is the most important to recognize when designing new bent caps. While an IRM may have been designed as such, depending on its condition in-service, it may be reclassified as an NSTM. Thus for design and fabrication, internally redundant members should be labeled as IRM and Fracture Control practice used, if they do not have load path redundancy.

The AASHTO *IRM Guide Specs* provide guidance for inspecting IRMs. They use a fatigue criterion in the faulted state to determine the interval for a "special inspection." This special inspection should not be confused with the traditional "hands-on" NSTM inspection conducted every 24 months as its purpose is to find severed components, not fatigue cracks within a component. The special inspection does not explicitly have to be "hands-on," but it does require sufficient rigor for an Inspector to identify a broken component. This rigor and how inspection intervals are determined would all be described in the Owner's written practice required by FHWA and may change from bridge to bridge.

Regardless of the bent cap being classified as an NSTM or IRM, members should be designed to allow full access, such as access holes on the outside of closed boxes and holes in the internal diaphragms of closed boxes, to adequately perform the required inspections.

6.2—REPAIR AND RETROFIT

In some cases, there may be good reasons for rehabilitating or retrofitting existing bent caps, ranging from time savings to transportation considerations. Bent caps, by their nature, are typically remarkably long and heavy members, which can make transporting them from the fabricator to the site a challenge. Transportation challenges and associated expenses may factor into a decision to retrofit rather than replace an existing bent cap. An existing bent cap may also be useful as a temporary support in a large project until a new construction phase is completed. That may require retrofit for the traffic conditions, support conditions, or special loads during construction. Below are some considerations when choosing whether to rehabilitate or retrofit an existing bent cap.

The primary traffic loads are point loads from the girders that the bent cap supports. In a rehabilitation or retrofit project, these loads may be in a different location from the original design of the bent cap. Specifically, stiffeners may need to be added under the revised girder locations and cover plates may need to be lengthened or added. Similarly, stiffeners above the bent cap bearings may need to be added if the bearing locations have been modified.

Bent caps that have been in service for long periods of time may have experienced some degree of section loss, repairs of which can be addressed like any other steel structural member. A significant consideration in the rehabilitation of bent caps is the fatigue life of the various components. Bent caps that have been in use for years may have connections at the end of their useful life, such as the welds for cover plates on the tension flange. One possible solution is to add bolts to replace the function of the welds. The bolts can transfer the shear through the members, and the welds at the end of their fatigue life are neglected. Other locations that might be critical are the stiffener welds that are below the neutral axis. Refer to AASHTO/NSBA G14.2*, Guidelines for Field Repairs and Retrofits of Steel Bridges* for further information.

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