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5.4 Pretensioned Prestressed Concrete Beams

The design of the standard BTB-BTE beam series is based on the 2017 AASHTO LRFD Bridge Design Specifications, 8th Edition.

5.4.1 PPCB

[AASHTO-LRFD Section 5]

5.4.1.1 General

The standard BTB-BTE beams and procedures described in this article meet AASHTO LRFD Specifications, 8th Edition, with minor modifications. The standard beams were designed using Bentley LEAP Bridge Concrete (LBC) CE V21 (Version 21.00.00.24). The designer should review related manual articles for decks [BDM 5.2], deck drains [BDM 5.8.4], railings [BDM 5.8.1], haunches [BDM 5.3], and bearings [BDM 5.7].

5.4.1.1.1 Policy overview

The Bureau-designed BTB-BTE standard bulb tee beams are available from approved fabricators. These standard beams are designed for simple span conditions and may be used for continuous span conditions without special analysis, provided that the use is within standard design and detailing conditions. The designer will need to check reinforcement in the deck at piers for nonstandard conditions [BDM 5.4.1.4.1.7]. Note that the use of the bulb tees in skewed structures may require wider abutment and pier caps to accommodate the wide flanges.

The Bureau assembles pretensioned prestressed concrete beam (PPCB) superstructures for short- to intermediate-span bridges from the standard beams. Each beam shape, BTB through BTE, is fabricated to a single depth in a series of standard lengths. Thus, the designer may select a beam shape to fit a range of spans between substructure components. With few exceptions, the maximum permissible beam spacing is 9.25 feet for the BTB-BTE beams. The complete set of standard beams is summarized in Table 5.4.1.1.1.

Table 5.4.1.1.1. Standard bulb tee pretensioned prestressed concrete beams designed for HL-93 loading

Beam Shape	BSB Standard Sheets		Span Range, feet	Depth, inches	Maximum Spacing feet
	Beams	Intermediate Diaphragms			
BTB	4750-4767	1036-1-BTB, 1036-2-BTB	30.00 to 105.00	36	9.25 (1)
BTC	4700-4719	1036-1-BTC, 1036-2-BTC	30.00 to 120.00	45	9.25 (1)
BTD	4730-4748	1036-1-BTD, 1036-2-BTD	50.00 to 135.00	54	9.25 (1)
BTE	4770-4790	1036-1-BTE, 1036-2-BTE	60.00 to 155.00	63	9.25 (1)

Table notes:

(1) Maximum spacing is reduced for the longest beams: BTB100, BTB105, BTC115, BTC120, BTD135, BTE150, and BTE155. See standard sheets for spacing limits.

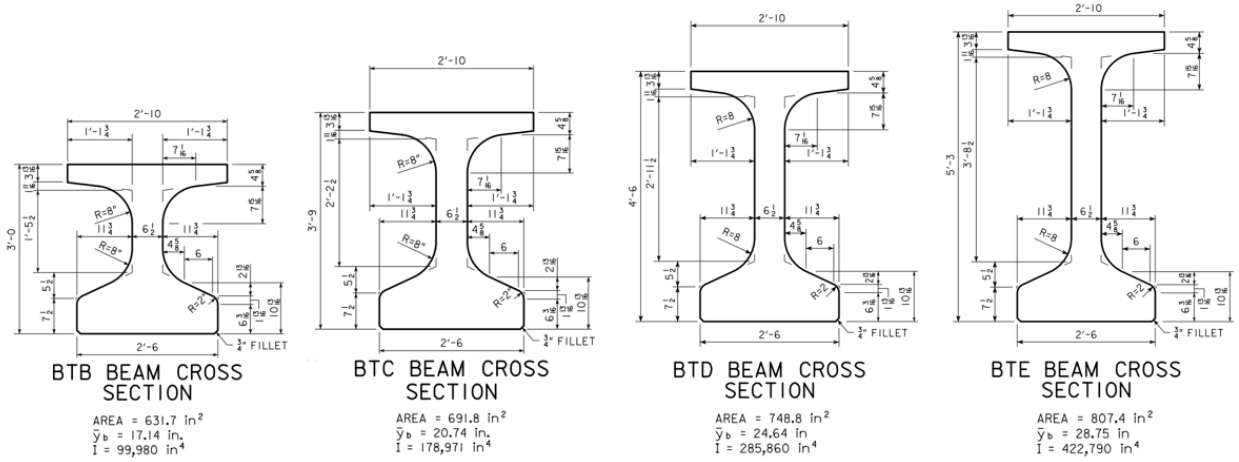


Figure 5.4.1.1.1. BTB to BTE beam shapes

Guidelines in this article [BDM 5.4.1] are provided to document the design criteria for the standard bulb tee beam shapes and for designing modified beams. When the designer changes the concrete strength, strand pattern, debonding requirements, or length of a standard beam, the designer shall designate the beam with an “S” to identify it as a beam that has been modified sufficiently to require special fabrication. For example, a BTD60 with a modified strand pattern should be designated SBTD60. The designer also shall note the special condition with the bid item. If a standard beam is changed in an unusual way, the designer should consult with the supervising Unit Leader for guidance.

At a minimum, designers shall use LBC software to check standard and non-standard PPCB designs used in projects. Standard LBC input files are available on the Iowa DOT website and may be used as a starting point in the checking process.

5.4.1.1.2 Design information

In the Bureau, for individually designed bridges the Preliminary Bridge Design Unit selects the standard bulb tee beam shape and locations of substructure components to fit the site and the spans available. The designer should check the preliminary design (or type, size, and location, commonly called the TS&L) with the goal of verifying the beam selection and determining the need for modifications. Occasionally a standard beam will need to be specially redesigned for nonstandard lengths, wide beam spacing, non-uniform beam spacing, or staged construction.

Considerable design and detail data for the standard bulb tee shapes are given on standard sheets [BDM Table 5.4.1.1.1]. It is Bureau policy to include in a set of bridge plans the applicable standard beam sheets corrected to remove all information that does not apply to the project.

The Iowa DOT Construction and Materials Bureau Instructional Memorandum “Precast and Prestressed Concrete Bridge Units” [CMB IM 570] and Appendices contain information on beam repair, cold weather placement, combined beam pours, and quality control. The memo is available from the Construction and Materials Bureau and is available on the Iowa Department of Transportation web site (http://www.iowadot.gov/erl/current/IM/Navigation/index_number.htm).

5.4.1.1.3 Definitions

[AASHTO-LRFD 3.3.2]

Standard bulb tee refers to the BTB-BTE beam shapes.

Unit Leader is the supervisor of the Bridges and Structures Bureau Preliminary Bridge Design Unit, Final Design Unit, or Consultant Coordination Unit.

5.4.1.1.4 Abbreviations and notation

[AASHTO-LRFD 3.3.2, 3.10.4.2]

BTB, BTC, BTD, BTE, standard bulb tee beam shapes
LBC, LEAP Bridge Concrete (formerly LEAP CONSPAN)**,** commercial software by Bentley for analysis and design of precast and prestressed beams for bridges.
d, distance from compression face to centroid of tension reinforcement
DC1, non-composite dead load of structural components and nonstructural attachments such as beams, deck, haunches, and diaphragms [AASHTO-LRFD 3.3.2].
DC2, composite dead load of structural components and nonstructural attachments such as barrier rails, sidewalks, curbs, and medians that are not part of the initial deck pour [AASHTO-LRFD 3.3.2].
DL, combined dead load
DW, dead load of wearing surfaces and utilities [AASHTO-LRFD 3.3.2].
$\mathbf{E_1}$, modulus of elasticity of material to be transformed to the concrete in a pretensioned prestressed concrete beam
$\mathbf{E_2}$, modulus of elasticity of concrete in a pretensioned prestressed concrete beam
$\mathbf{f'_c}$, specified 28-day concrete compressive strength
$\mathbf{f'_{ci}}$, specified compressive strength of concrete at time of initial prestress
GUTS, guaranteed ultimate tensile strength
IM, dynamic load allowance (also live load impact)
L, span length center to center of bearings
LL, live load
LRFD, load and resistance factor design

n, modular ratio, E_1/E_2
PPCB, pretensioned prestressed concrete beam
S, notation for a modified standard bulb tee beam
S_{D1}, horizontal response spectral acceleration coefficient at 1.0-sec. period modified by long-period site factor [AASHTO-LRFD 3.10.4.2]
TS&L, type, size, and location
VPD, vehicles per day, which includes traffic in both directions

5.4.1.1.5 References

American Society for Testing and Materials (ASTM). "A416/A416M-18, Standard Specification for Low-Relaxation Seven-Wire Steel Strand for Prestressed Concrete." ASTM International. West Conshohocken, PA.

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Precast/Prestressed Concrete Institute (PCI). *PCI Design Handbook, Eighth Edition.* Chicago: PCI, (MNL-120-17) 2021.

Precast/Prestressed Concrete Institute (PCI). *Recommended Practice for Lateral Stability of Precast, Prestressed Concrete Bridge Girders*. Chicago: PCI, (CB-02-16) 2016.

5.4.1.2 Loads

There are many loads applied to the superstructure and transmitted to the substructure. The articles that follow discuss the typical loads that affect the design of the superstructure, but the designer shall consider the full range of loads. The designer should consult the pier article [BDM 6.6] for a more complete discussion of loads.

5.4.1.2.1 Dead

The Bureau classifies dead load as follows.

- DC1 is noncomposite dead load including beams, deck, haunches, and diaphragms.

- DC2 is composite dead load including barrier rails, sidewalks, curbs, and medians that are not part of the initial deck pour.
- DW is composite dead load of wearing surfaces and utilities.

The deck weight that is part of DC1 shall be distributed to each beam assuming that the deck between beams is simply supported and all the deck weight of an overhang is distributed to the exterior girder.

For a typical PPCB bridge the Bureau requires a future wearing surface load of 0.020 ksf, which is part of the DW load.

For two course decks the designer shall consider the second course as DC2 load, rather than DW load, because it will be placed under relatively controlled conditions by the bridge contractor, therefore the use of the lower DC2 load factor is justified.

For superstructures with roadway widths no greater than 44 feet, DC2 and DW shall be distributed equally to all beams.

For superstructures with roadway widths greater than 44 feet, the future wearing surface shall be distributed equally to all beams. Each barrier rail and raised sidewalk cast after the deck along the edge of the superstructure shall be distributed one-half to the exterior beam, one-quarter to the first interior beam, and one-quarter to the second interior beam. Note that the preceding policy is different from the policy for design of pier cap cantilevers [BDM 6.6.2.1].

The standard BTB-BTE beams were designed using the LBC software. The input and output files for each standard beam design are posted on the BSB website. Each standard beam design satisfies both exterior and interior beam design requirements and may be used in either location for bridges meeting the standard design assumptions.

- Most of the BTB-BTE beam designs are based on a superstructure cross-section model with an out-to-out deck width of 44 feet, roadway width 40.834 feet, a 5-beam cross-section with 9.25 feet beam spacings, and 3.50 feet deck overhangs. Some of the longest spans for the BTB-BTE shapes required narrower beam spacings and had the geometrical input adjusted accordingly.
- The distance from gutter line to edge of deck is 1.583 feet for the BTB-BTE beams.
- The DC1 deck load is distributed to each beam as described previously. The deck load for interior portions of the deck is based on an 8.50-inch thickness. The deck load for the exterior overhangs is based on a 10-inch thickness at the edge of deck and an 11-inch thickness at the edge of the top flange.
- The DC1 haunch load is based on a 1.50-inch thick haunch for the BTB-BTE beams.
- Intermediate steel diaphragms are located along the span per BDM 5.4.1.4.2 with each diaphragm resulting in a 0.500 kip DC1 point load for the BTB-BTE interior beams. Diaphragm loads for exterior beams are half the interior beams.
- A DC2 rail weight of 0.590 klf, based on the 44-inch tall MASH TL-5 single slope rail, was conservatively applied. The rail weight was distributed such that each beam in the cross-section carries the equivalent of one-half a rail. This conservative approach was incorporated to better address narrow bridges and staged construction scenarios.
- The FWS load of 0.020 ksf is applied over the roadway width and is equally distributed to all 5 beams in the cross-section as a DW load.

5.4.1.2.2 Live

[AASHTO-LRFD 3.6.1.2, C3.6.1.2.1, 3.6.1.3, C3.6.1.3.1, 3.6.1.6, 4.6.2.2]

Unless special requirements govern the design, vehicular live load (LL) for PPCB superstructures shall be HL-93 [AASHTO-LRFD 3.6.1.2]. It shall be applied as discussed in the AASHTO LRFD Specifications [AASHTO-LRFD 3.6.1.3].

The Strength II load combination which relates to owner-specified special design vehicles and evaluation permit vehicles is not ordinarily considered [AASHTO-LRFD 3.4.1]. In unusual situations identified by the Chief Structural Engineer, where a bridge will experience a high percentage of truck traffic, accumulation of trucks due to traffic flow control, or special industrial loads, the designer will need to consider additional loading as discussed in the AASHTO LRFD Specifications commentary [AASHTO-LRFD C3.6.1.2.1, C3.6.1.3.1]. In these situations, the designer shall consult with the Chief Structural Engineer.

Sidewalk and other live loads shall be added to the superstructure live load when applicable. A pedestrian load (PL) of 0.075 ksf shall be applied to all sidewalks [AASHTO-LRFD 3.6.1.6] and considered simultaneously with the vehicular live loads.

Distribution of live load to interior and exterior beams shall be as given in the AASHTO LRFD Specifications [AASHTO-LRFD 4.6.2.2]. The interior and exterior beams shall typically have the same capacity and any exceptions must be approved by the Chief Structural Engineer. For skewed bridges the designer shall use the shear increase [AASHTO-LRFD 4.6.2.2.3c] but not the moment reduction [AASHTO-LRFD 4.6.2.2.2e]. The designer shall not use the analysis based on the conventional approximation of loads on piles when determining exterior beam live load distribution factors for prestressed beam bridges [AASHTO-LRFD 4.6.2.2.2d and Commentary]. The standard BTB-BTE beams were designed for a shear increase based on a 60-degree skew [AASHTO-LRFD 4.6.2.2.3c].

5.4.1.2.3 Dynamic load allowance

[AASHTO-LRFD 3.6.2.1]

The static effects of the design truck or tandem, other than centrifugal and braking forces, shall be amplified by the dynamic load allowance (IM) given in the AASHTO LRFD Specifications [AASHTO-LRFD 3.6.2.1].

5.4.1.2.4 Earthquake

[AASHTO-LRFD 3.10.3.1, 3.10.4.2, 3.10.6, 3.10.9.2, 4.7.4.1, 4.7.4.4]

Based on the acceleration coefficient S_{D1}, all of Iowa with Site Class A through E shall be classified as Seismic Zone 1 [AASHTO-LRFD 3.10.3.1, 3.10.4.2, 3.10.6]. Thus, for typical bridges no seismic loading (EQ) or analysis is required [AASHTO-LRFD 4.7.4.1]. However, for unusual projects such as bridge sites determined to be Site Class F and for Missouri River and Mississippi River bridges the designer shall determine the seismic zone and perform seismic analysis as required by the AASHTO LRFD Specifications.

Typical continuous PPCB bridges with integral abutments, standard fixed pier connections, and/or standard steel reinforced elastomeric bearings do not require design for seismic connection forces. However, the designer should review bridges with deck expansion joints, bronze plate bearings, fixed shoe bearings, rocker bearings, and special bearings for the need to design for seismic connection forces [AASHTO-LRFD 3.10.9.2 and BDM 5.7.2.6]. At deck expansion joints there also will be a need to check bearing seat widths [AASHTO-LRFD 4.7.4.4].

5.4.1.2.5 Construction

[AASHTO-LRFD 6.10.3]

For a typical superstructure constructed with the standard bulb tee beams, the designer will not need to check constructability. The contractor is responsible for ensuring stability of beams from the time of beam placement up through the deck reaching full 28-day strength [BDM CADD NOTE E202]. The standard bulb tee beams were checked for the following construction loads on the exterior beam. The loading condition assumes the wet deck concrete has been poured from one end of the span to midspan. (Note that a loading condition assuming the wet deck concrete has been poured over the full span length was determined not to control the constructability checks.) All vertical loads were applied to the non-composite

exterior beam and any effects due to load eccentricity were not included. The Service I, Service III, and Strength I load combinations were considered. Load factors for the Service I and Service III load combinations are 1.00 whereas the load factors for the Strength I load combination are listed with each bullet.

- Dead load of exterior beam using a 1.25 load factor.
- Dead load of wet deck and haunch concrete to exterior beam [BDM 5.4.1.2.1] using a 1.25 load factor. This load is applied from the start of the span to midspan.
- Dead load of permanent diaphragm(s) to exterior beam [BDM 5.4.1.2.1] using a 1.25 load factor.
- Dead load of forms: 0.010 ksf on the tributary area of the deck width with a 1.50 load factor. This load is applied to the full span length.
- Dead load of edge rail and walkway at the edge of the deck form: 0.075 klf [IDOT SS 2403.03, L, 4, d.] with a 1.25 load factor. This load is applied to the full span length.
- Construction live load on walkway: 0.050 ksf [IDOT SS 2403.03, L, 4, e.] on a 2'-4" wide walkway with a 1.50 load factor. This load is applied to the full span length.
- Construction live load on deck: 0.050 ksf [IDOT SS 2403.03, L, 4, e.] on the tributary area of the deck width with a 1.50 load factor. This load is applied from midspan to end of span.
- Live load of finishing machine located along the edge of the deck form: 9.4 kips with a 1.50 load factor. The load is placed as 4 individual point loads of 2.35 kips each. The point loads are spaced as follows 2'-4'-2' with the second point load located at the center of the span.

For any unusual conditions, the designer shall check construction loads [AASHTO-LRFD 2.5.3, 3.4.2]. The designer shall consider the deck pouring sequence in the design of the beams for constructability (see BDM 5.2.4.1.2). Typically, during construction the exterior beam supports the deck overhang by means of an overhang bracket. The bracket transfers to the exterior beam both vertical and lateral loads that are dependent on the bracket shape. As part of a constructability check the designer shall consider these construction loads to the exterior beam. A Strength III combination with wind loads [IDOT SS 2403.03, L, 4, g.] may also be appropriate.

For most bridge projects it is assumed that construction can take place without cranes, construction vehicles, construction equipment, and concentrated quantities of construction materials on the bridge in addition to the loads listed above. If, however, the contractor does need to place additional loads on the bridge larger than those permitted by the Standard Specifications [IDOT SS 1105.12, D], the contractor will be required to submit structural analysis by an Iowa-licensed engineer for approval. Thus, the bridge designer may be required to review construction loading after letting of the bridge contract.

5.4.1.3 Load application to superstructure

5.4.1.3.1 Load modifier

[AASHTO-LRFD 1.3.2, 3.4.1]

Load factors shall be adjusted by the load modifier, which accounts for ductility, redundancy, and operational importance [AASHTO-LRFD 1.3.2, 3.4.1]. For typical PPCB bridges with at least three parallel beams the load modifier shall be taken as 1.0. For typical staged construction scenarios the load modifier may be taken as 1.0 when two or more beams carry staged traffic.

5.4.1.3.2 Limit states

[AASHTO-LRFD 2.5.2.6.2, 2.5.3, 3.4.1, 3.4.2, 5.5.3.1, 5.9.2.2, 5.9.2.3.1, 5.9.2.3.2]

The standard beams have been checked for compression and tension stresses at transfer (before losses) [AASHTO-LRFD 5.9.2.3.1]. The designer shall perform those checks for modified beams.

For the typical PPCB bridge superstructure, the designer shall consider the following limit states after losses for beam design [AASHTO-LRFD 3.4.1].

- Strength I, superstructure with vehicles but without wind

- Service I, superstructure with vehicles (Although the Service I limit state includes design 3-second gust wind speed at 70 mph, wind normally is not included in beam design under this limit state. In unusual cases where wind on a beam or PPCB superstructure should be investigated the designer shall use the Strength III and Strength V limit states.)
- Service III, tension in the prestressed concrete superstructure with respect to crack control [The Service III limit state load factor for live load (γ_{LL}) was taken as 0.80 for the design of the standard PPCB beams (AASHTO-LRFD Table 3.4.1-4). Note that Iowa has historically used and continues to use the approximate estimate of time-dependent losses as found in AASHTO LRFD 5.9.3.3 in conjunction with transformed section properties.]
- Fatigue

Usually Strength III (superstructure without vehicles but with design 3-second gust wind speed at 115 mph), Strength IV (superstructure with very high dead load to live load ratios), and Strength V (superstructure with vehicles and with design 3-second gust wind speed at 80 mph) will not control for the typical completed bridge, and the designer may use experience with typical PPCB bridges to avoid detailed checking of these three limit states. For long span and other non-typical bridges, the designer shall fully consider all appropriate limit states.

For the standard beams in a typical bridge, or for modified beams designed as fully prestressed members with tensile stresses within the limits of Service III limit state, the designer need not investigate the fatigue limit state [AASHTO-LRFD 5.5.3.1].

The designer shall consider all appropriate service and strength limit states during construction [AASHTO LRFD 2.5.3, BDM 5.4.1.2.5].

Load combinations and load factors for limit states and stress limits for service limit states are given in the AASHTO LRFD Specifications [AASHTO-LRFD 3.4.1, 3.4.2, 5.9.2.2, 5.9.2.3.2].

5.4.1.4 BTB-BTE beams

Analysis, design, and detailing information in this article is provided primarily to document the conditions under which the standard bulb tee beams were designed and to provide guidelines for design of modified beams. However, for the standard beams some of the information will be useful, such as the information under analysis assumptions, for applying superstructure loads to the substructure.

5.4.1.4.1 Analysis and design

[AASHTO-LRFD Section 5]

The designer shall use the LRFD method for design of modified standard beams and superstructures [AASHTO-LRFD Section 5].

For PPCB bridge deck guidelines, see the moment article [BDM 5.4.1.4.1.7].

For diaphragms, see the detailing article [BDM 5.4.1.4.2].

5.4.1.4.1.1 Analysis assumptions

[AASHTO-LRFD 5.9.4.3.1]

For service and strength limit states, a standard bulb tee is checked conservatively in a simple span condition for all loads and is considered noncomposite for DC1 and composite for DC2, DW, live load, and dynamic load allowance. As the PPCB superstructure is assembled in the field, the addition of the deck changes the structural behavior of the beams. The continuity resulting from placement of the deck affects both deflection of the beams and the loads transmitted to the substructure. For the standard shapes the Bureau makes the span and load distribution assumptions given in Table 5.4.1.4.1.1.

Table 5.4.1.4.1.1. Assumptions for design of BTB-BTE beams and loads to the substructure

Load	Theoretical Span and Composite Conditions	Bureau Practice Span and Composite Conditions			
		Beam Strength and Service Limit States	Beam Deflections	Abutment Loads	Pier Loads
DC1	Simple Noncomposite	Simple Non-composite	Simple Non-composite	Simple Non-composite	Simple Non-composite
DC2 and DW(1)	Continuous Composite	Simple(2) Composite	Continuous Composite	Continuous Composite	Continuous Composite(3)
Live Load and Dynamic Load Allowance	Continuous Composite	Simple(2) Composite	Continuous Composite	Continuous Composite	Continuous Composite(3)

Table notes:

(1) For beam, bearing, and pier frame design, DW and DC2 are distributed equally to all beams for roadway widths less than or equal to 44 feet. For wider bridges, the future wearing surface (DW) is distributed equally to all beams, and each barrier rail load (DC2) is distributed half to the exterior beam and one-quarter to each of the adjacent two interior beams [BDM 5.4.1.2.1]. A different, conservative policy applies for pier cap cantilever design. The future wearing surface is distributed equally to all beams, and all barrier rail load is distributed only to the exterior beams [BDM 6.6.2.1].

(2) Bureau policy is to design beams using the simple span condition for all strength and service stress checks and add longitudinal reinforcement to the concrete deck above continuous pier supports to avoid deck joints and control tension cracking. The intent is to meet the exception discussed in the AASHTO LRFD Specifications for simple spans detailed for continuity [AASHTO LRFD 5.12.3.3.1, C5.12.3.3.1]. However, the longitudinal reinforcement and continuity diaphragms will cause the superstructure to behave approximately as a continuous structure, and that condition should be assumed for deflections and abutment and pier loads.

(3) Moment of inertia is to be taken as constant over the length of the bridge.

Prestress is assumed to be fully effective at the transfer length of 60 strand diameters from the point where bonding commences which is at the end of a pretensioned beam for fully bonded strands and at the end of the debonding length for a partially debonded strand [AASHTO-LRFD 5.9.4.3]. For the usual 0.6-inch diameter strand, 60 diameters is a distance of 36 inches.

For design of BTB-BTE beams the deck haunch is assumed to be 1.50 inches for weight and 0.50 inch for composite section calculations.

For the design of nonstandard beam spacings or arrangements the distance from gutter to centerline of exterior beam should be set at 1.92 feet for BTB-BTE beams.

5.4.1.4.1.2 Materials

[AASHTO-LRFD C5.4.2.4, 5.4.3.2, 5.4.4.2]

Concrete strengths for the standard pretensioned prestressed concrete beams shall be as specified in Tables 5.4.1.4.1.2-1.

Table 5.4.1.4.1.2-1. Concrete specifications for standard BTB-BTE pretensioned prestressed concrete beams

Beam Type	BSB Standard Sheet	f'_{ci}, ksi (1)	f'_c, ksi	
			Beam (2)	Deck
BTB30-BTB65	4750	4.5	5.0	4.0
BTB70	4750	5.0	5.5	4.0
BTB75	4750	5.5	6.5	4.0
BTB80	4750	6.0	7.0	4.0
BTB85	4750	6.5	7.5	4.0
BTB90	4750	7.5	8.5	4.0
BTB95	4750	8.0	9.5	4.0
BTB100	4765	8.0	10.0	4.0
BTB105	4765	8.5	10.0	4.0
BTC30-BTC60	4700	4.5	5.0	4.0
BTC65-BTC75	4700	5.0	6.0	4.0
BTC80-BTC90	4700	6.0	7.0	4.0
BTC95	4700	6.5	7.5	4.0
BTC100	4700	7.0	8.0	4.0
BTC105	4700	7.5	8.5	4.0
BTC110	4700	7.5	9.0	4.0
BTC115	4700	7.5	9.5	4.5
BTC120	4719	8.0	10.0	4.5
BTD50-BTD70	4730	4.5	5.0	4.0
BTD75-BTD85	4730	5.0	6.0	4.0
BTD90-BTD95	4730	5.5	6.5	4.0
BTD100-BTD105	4730	6.0	7.0	4.0
BTD110	4730	6.5	7.5	4.0
BTD115-BTD120	4730	7.0	8.0	4.0
BTD125	4730	7.5	8.5	4.0
BTD130	4730	7.5	9.0	4.0
BTD135	4748	8.0	9.5	4.0
BTE60-BTE75	4770 (Sheet 1 of 2)	4.5	5.0	4.0
BTE80-BTE110	4770 (Sheet 1 of 2)	5.0	6.0	4.0
BTE115-BTE120	4770 (Sheet 1 of 2)	5.5	6.0	4.0
BTE125-BTE130	4770 (Sheet 1 of 2)	6.5	7.5	4.0
BTE135	4770 (Sheet 1 of 2)	7.0	8.0	4.0
BTE140	4770 (Sheet 1 of 2)	7.5	8.5	4.0
BTE145	4770 (Sheet 1 of 2)	7.5	9.0	4.0
BTE150	4770 (Sheet 1 of 2)	7.5	9.5	4.5
BTE155	4790 (Sheet 1 of 2)	8.0	10.5	4.5

Table notes:

(1) For modified beams, the designer should generally try to limit f'_{ci} to 7.5 ksi or less but may increase f'_{ci} up to 8.0 ksi as needed. An f'_{ci} of 8.5 ksi is only used for the BTB105 and may only be used for other beam designs with the approval of the Chief Structural Engineer.

(2) For modified beams, f'_c shall normally be limited to 10.0 ksi. With approval of the supervising Unit Leader, f'_c may be increased to 10.5 ksi.

The BTB-BTE beams were designed to try and limit f'_{ci} to 7.5 ksi or less by first using deflected strands and then debonding strands if needed. Concrete release strengths of 7.5 ksi or less allow for more efficient beam production since less time is required to remove the beams from the casting beds so that

production can begin on a new set of beams. For some beams an f'_{ci} of 8.0 ksi was required to meet the minimum standard beam spacing requirements for the standard roadway widths. The standard BTB105s required an f'_{ci} of 8.5 ksi to meet the minimum standard beam spacing requirements for the standard roadway widths.

The modulus of elasticity for pretensioned prestressed concrete beams shall be based on a concrete weight of 0.150 kcf. This weight is larger than given in the AASHTO LRFD Specifications [AASHTO-LRFD C5.4.2.4] to account for the higher density of precast beam concrete.

For the usual Class C deck concrete the 28-day compressive strength shall be taken as 4.0 ksi unless the beam design requires a higher strength. Reinforcing steel shall be Grade 60, with a minimum yield strength of 60 ksi. Modulus of elasticity for reinforcing shall be taken as 29,000 ksi [AASHTO-LRFD 5.4.3.2].

Prestressing strands shall be ASTM A416 Grade 270 low relaxation, with the properties listed in Table 5.4.1.4.1.2-3.

Table 5.4.1.4.1.2-3. Low relaxation prestressing strand data for design

Nominal Diameter, inches	Nominal Area, in^2	Ultimate Tensile Strength, kips	Maximum Initial Prestress (1), kips	Modulus of Elasticity (2), ksi
½ (3)	0.153	41.300	---	28,500
0.6	0.217	58.600	43.950	28,500

Table notes:

(1) The maximum initial prestress value in the table for design is 75% of guaranteed ultimate tensile strength (GUTS). The Bureau prefers that the limit be set lower, at approximately 72.6% of GUTS, so that the fabricator can make adjustments that do not require additional cost for stressing strands at cold temperatures. During fabrication Iowa DOT policy allows for temporary prestressing increases to a maximum stress not exceeding 80% of the minimum ultimate tensile strength [IDOT SS 2407.03, G, and AASHTO-LRFD-CS 10.10.2].

(2) For design of the standard beams the Bureau has used this modulus of elasticity suggested in the AASHTO LRFD Specifications [AASHTO-LRFD 5.4.4.2]. For cold weather stressing adjustments, a fabricator may use a value based on data supplied by the strand manufacturer [CMB IM 570, Appendix C].

(3) These ½-inch strands are to be used for lifting loops only. All stressed prestressing strands in the standard beams shall be 0.6 inches in diameter.

5.4.1.4.1.3 Design resistance and stress limits

[AASHTO-LRFD 5.9.2.3.1, 5.9.2.3.2]

Design resistance shall be determined in accordance with the AASHTO LRFD Specifications.

Temporary stress limits for beams at transfer (before losses) shall be as follows [AASHTO-LRFD 5.9.2.3.1].

- Compression: $0.65f'_{ci}$ [AASHTO-LRFD 5.9.2.3.1a]
- Tension: $0.24\lambda(f'_{ci})^{0.5}$ where $\lambda = 1.0$ for normal weight concrete [AASHTO-LRFD Table 5.9.2.3.1b-1]. All standard beams shall satisfy the requirements for bonded reinforcement to resist the tensile force in the concrete assuming an uncracked section.

Stress limits for service limit states (after losses) shall be as follows [AASHTO-LRFD 5.9.2.3.2].

- Compression, Service Limit State Load Combination I: [AASHTO-LRFD Table 5.9.2.3.2a-1]
  - $0.45f'_c$ for PS + DL
  - $0.60\phi_w f'_c$ for PS+DL+(LL+IM) where $\phi_w = 1.0$
    Note: DL includes DC1, DC2, and DW loads for both cases.

- Tension, Service Limit State Load Combination III: $0.19\lambda(f'_c)^{0.5} \leq 0.60$ ksi where $\lambda = 1.0$ for normal weight concrete [AASHTO-LRFD Table 5.9.2.3.2b-1]. All standard beams are assumed to be subject to not worse than moderate corrosion conditions.

Stress limits for fatigue limit state (after losses) shall be as follows [AASHTO-LRFD 5.5.3].

- Compression, Fatigue Limit State Load Combination I: [AASHTO-LRFD 5.5.3.1]
  - $0.40f'_c$ for $0.50*(PS + DL)+LL_{fatigue}$

Stress limits for noncomposite beams (after losses) due to construction loads taken from BDM 5.4.1.2.5.shall be as follows.

- Compression, Service Limit State Load Combination I: $0.45f'_c$ [AASHTO-LRFD Table 5.9.2.3.2a-1]
- Tension, Service Limit State Load Combination III: No tension in the bottom flange.

5.4.1.4.1.4 Section properties

[AASHTO-LRFD 4.6.2.6.1, 5.6.1]

For calculating stresses at the service limit state in a pretensioned prestressed concrete beam the designer should utilize the transformed section including all fully stressed strands. For any partially debonded straight strands, the portion of the strand that is debonded at the beam ends should not be considered in the transformed section properties over the debonded length because the debonded strand is not composite with the beam. Strands should be transformed to PPCB concrete using the modular ratio for each steel material [AASHTO-LRFD 5.6.1].

For estimating camber using LBC, the designer shall use the transformed section option. Do not transform mild longitudinal bars or strands stressed to not more than 5 kips which are substituted for mild longitudinal bars in the top flange. LBC lumps total strand area at the center of gravity of the strands when determining transformed section properties.

For deflection computations the designer should use properties of the transformed cross section.

For determining composite section properties the deck thickness shall be reduced by 0.75 inches to account for the built-in wearing surface. The effective deck width shall be determined in accordance with AASHTO LRFD Specifications for effective flange width [AASHTO-LRFD 4.6.2.6.1] and considering the reduced deck thickness.

For calculating stresses in composite sections in positive moment regions the designer shall transform the deck concrete (but not the reinforcement) to beam concrete using the modular ratio for deck concrete and the effective flange width for tee beams [AASHTO-LRFD 4.6.2.6.1]. Within negative moment regions the designer shall consider only the beam section and deck reinforcing to be effective in resisting negative moment.

5.4.1.4.1.5 Deflected strands and debonded straight strands

Deflected (harped) strands and partially debonded (shielded) straight strands are used primarily to keep the stresses at the ends of a beam within limits at transfer while utilizing increased prestressing force in the beam. Deflected strands and partially debonded straight strands generally need to be used only in longer beams. Deflected strands and partially debonded straight strands may be used in the same beam, but Iowa prefers to first use deflected strands before including partially debonded straight strands. The number of deflected strands generally should be limited to 25 to 33% of the total number of strands. Debonding limits for strands shall be based on AASHTO LRFD 5.9.4.3.3 from the 2017, 8th edition and supplemented by 2020, 9th edition as follows.

- The number of partially debonded strands should not exceed 25% of the total number of strands.
- The number of debonded strands in any horizontal row shall not exceed 40% of strands in that row.

- Debonding shall not be terminated for more than six strands in any given section. When a total of ten or fewer strands are debonded, debonding shall not be terminated for more than four strands in any given section.
- Longitudinal spacing of debonding termination locations shall be at least 60 diameters apart.
- Debonded strands shall be symmetrically distributed about the vertical centerline of the member cross-section. Debonding shall be terminated symmetrically at the same longitudinal location.
- Alternate bonded and debonded strand locations both horizontally and vertically.
- The debonding length from the beam end should be limited to 20% of the span length or one-half the span length minus the development length, whichever is less.
- Bond all strands within the horizontal limits of the web.
- Position debonded strands furthest from the vertical centerline except bond the outer-most strands in all rows.
- Deflected strands shall be fully bonded.

Deflected strands increase the shear strength near beam ends and should be considered when determining shear reinforcement. Partially debonded straight strands at the beam ends decrease the shear strength near the beam ends and should be considered when determining shear reinforcement.

Deflected strands in the standard bulb tee beams are shown on standard sheets with hold down points at 0.40L from each beam end. The beam fabricator has the option of moving the hold down points 0.05L toward the beam ends. Therefore, for modified beams the designer shall investigate beam stresses for hold down points at each of the 0.35L and 0.40L locations. At a beam end the center of gravity of the deflected strand group is permitted a vertical tolerance of $\pm$ 1 inch. The designer shall investigate beam stresses for the strand group at each of the extreme tolerances.

The Bureau prefers to use a minimum debonding length of 6 feet for each debonded strand at the beam ends to better verify through measurement that debonding has been achieved at the fabrication plant per CMB IM 570. However, for the BTB105 beams it was decided to allow a 3-foot debonded strand length for a portion of the debonded strands and use 6 feet for the remaining debonded strands since by code not all strands could be debonded at the same section. Designers shall consult with the Chief Structural Engineer when debonding lengths shorter than 6 feet are required by design.

Fabricators prefer not to debond any strands in the bottom row to avoid contact between the sheathing and the stirrups as contact may compromise the sheathing. Check with the Chief Structural Engineer prior to specifying debonding any strands in the bottom row.

Partially debonded straight strands at beam ends are included for standard beams BTB95, BTB100, BTB105, BTC115, BTC120, BTD130, BTD135, and BTE150. CMB IM 570 specifies the requirements for the plastic slit sheathing used to achieve partial strand debonding at the beam ends.

5.4.1.4.1.6 Prestress losses

[AASHTO-LRFD 5.9.3]

Prestress losses shall be determined in accordance with specific AASHTO provisions for elastic shortening, creep, shrinkage, and relaxation [AASHTO-LRFD 5.9.3]. The designer shall use the approximate method for the time-dependent losses [AASHTO-LRFD 5.9.3.3].

When the designer transforms strands to beam concrete, the designer should neglect loss of concrete stress due to elastic shortening because the loss automatically will be included in the computations. The elastic shortening loss in the strands, however, shall be computed and considered, as applicable, for all other prestress losses.

5.4.1.4.1.7 Moment

[AASHTO-LRFD 5.6.3.3]

For service limit state checks, beam stresses shall be computed from moments determined for span and composite conditions given under analysis assumptions [BDM Table 5.4.1.4.1.1]. Beam stresses shall be checked under transfer and service limit states at all critical sections including the following:

- Transfer (60 strand diameters from beam end),
- Hold-down for deflected strands, and
- Midspan.

Factored flexural resistance in the positive moment regions for the Strength I Limit State check and constructability check is determined in accordance with AASHTO-LRFD 5.6, however, the strain compatibility approach [AASHTO-LRFD 5.6.3.2.5] is not used for the standard beam designs.

The minimum amounts of prestressing and reinforcing steel in beams shall meet the limits in the AASHTO LRFD Specifications [AASHTO-LRFD 5.6.3.3]. The limits for maximum amounts of prestressing and reinforcing steel in beams were removed from AASHTO in 2005, with the effects of an over-reinforced (compression-controlled) section accounted for by a reduction in the resistance factor.

Where the superstructure is detailed for continuity above a pier, reinforcement in the deck shall be designed for Strength I negative moments due to live load with impact, future wearing surface, curbs and rails, and any other load applied after the deck has cured. Note, however, the exception in (3) below. Moments due to creep and shrinkage after continuity is achieved shall not be considered.

Standard sheets for PPCB bridge decks [BSB SS 4380-BTB-4 to 4385-BTE-6, 4556-BTC-4 to 4561-BTE-6] have the correct negative moment reinforcing for the span lengths available with standard PPCBs. Per BDM 5.8.5.3 when 60 ksi No. 9 bars are required in the top and bottom of the deck in the negative moment regions the designer has the option to substitute 80 ksi No. 8 bars to reduce congestion. For typical bridges that make use of the deck standard sheets the designer need not check deck reinforcement, but for greater spans or wider, nonstandard beam spacings the designer shall check the need for additional deck reinforcement and add reinforcement as necessary.

In cases where it is necessary to check deck reinforcement above piers the designer shall proceed as follows:

(1) Determine the distribution steel and spacing.
(2) Add negative moment reinforcement between the distribution steel. Start by adding steel to the top layer, and add to the bottom layer only if more steel is required. See Figure 5.4.1.4.1.7.
(3) For negative moment reinforcement use bars no larger than #9. In extreme cases this bar size limit and the placement limits in (2) may result is less steel than required for Strength I negative moment, and the Bureau will accept that condition on a case-by-case basis. Contact the Chief Structural Engineer for approval.
(4) Terminate the top and bottom layer negative moment reinforcement at the eighth points of the spans, which generally will be the locations of permissible concrete deck construction joints.
(5) Do not check concrete compression due to continuity.
(6) For unusual superstructures contact the Chief Structural Engineer.

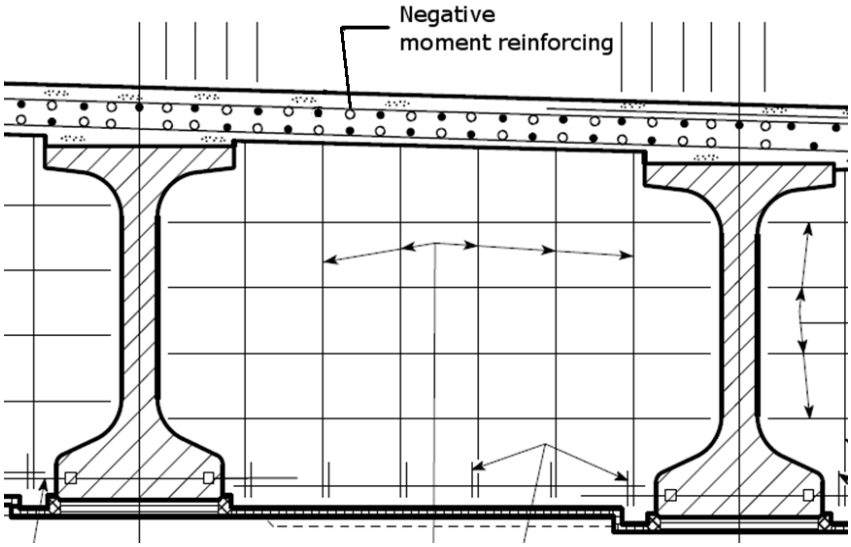


Figure 5.4.1.4.1.7. Negative moment reinforcement above piers

5.4.1.4.1.8 Shear

[AASHTO-LRFD 5.7.3, 5.7.4]

Web shear reinforcement shall be designed in accordance with AASHTO LRFD Specifications [AASHTO-LRFD 5.7.3]. The vertical component of the deflected strand prestressing force should be included, if present, when calculating beam shear strength. Partially debonded straight strands at the beam ends decrease the shear strength near the beam ends and should be considered, if present, when calculating beam shear strength.

For a standard beam, Bureau practice is to gradually increase web shear reinforcement spacing from the anchorage zones to points at about 0.25 to 0.35 of the beam length and then use a constant spacing, as determined by design requirements, for the center portion of the beam.

The designer shall investigate horizontal shear between the precast beam and cast-in-place deck and shall provide tie reinforcement in accordance with AASHTO LRFD Specifications [AASHTO-LRFD 5.7.4]. The total interface shear considered in the design of the standard beams includes contributions from all DC1, DC2, DW, and LL loads. The top flange of the beam is to be struck off level and intentionally roughened in the transverse direction to a full amplitude of 0.25 inch. ~~except for a 2-inch wide finish along one edge of the flange [CMB IM 570]. The strip with the smooth finish is intended to aid in taking elevations during construction.~~ The standard beams shall conservatively deduct~~ed~~ an assumed 2-inch wide smooth finish along both edges of the flange when evaluating interface shear.

5.4.1.4.1.9 Deflection and camber

[AASHTO-LRFD 2.5.2.6.2, 2.5.2.6.3, 3.6.1.1.2]

The minimum beam depth plus haunch plus composite deck thickness (less wearing surface) generally should be at least one twenty-fifth of the span length (0.04L) [AASHTO-LRFD 2.5.2.6.3] for continuous spans. Span lengths for BTB beams meet this criterion to 91 feet, BTC beams meet this criterion to 110 feet, BTD beams meet this criterion to 129 feet, and BTE beams meet this criterion to 147 feet [BDM Table 5.4.1.1.1]. Although the longest bulb tee beams fail to meet the depth criterion, they are adequate for the load and maximum spacing conditions under which they were designed. Other shallow beam conditions shall be approved by the supervising Unit Leader.

The Bureau requires the designer to check deflection according to the AASHTO LRFD criteria [AASHTO-LRFD 2.5.2.6.2]. When computing deflections the designer shall use the number of 12-foot wide design traffic lanes that fit on the roadway with the applicable multiple presence factor for three or more lanes [AASHTO-LRFD 3.6.1.1.2]. Live load plus dynamic load allowance shall be distributed equally to all beams.

Preferably the deflection for live load plus dynamic load allowance should not exceed L/1000, but in no case shall the deflection exceed L/800. For bridges that carry pedestrian traffic the deflection for live plus dynamic load allowance shall not exceed L/1000. If live plus dynamic load allowance deflection controls the design, the designer shall consult with the supervising Unit Leader.

For estimating deflection and camber, the designer should use the section properties recommended in a previous article [BDM 5.4.1.4.1.4].

Iowa State University (ISU) has completed research addressing camber prediction and camber measurement. BDM 5.4.1.1.5 contains the research report reference. Changes to initial (at-release) camber and final (at-erection or after-losses) camber due to recommendations from the research report are incorporated into the PPCB standard sheets.

The flowchart in Figure 5.4.1.4.1.9 illustrates the camber deflection multipliers applied to the theoretical instantaneous camber calculation from LBC in order to determine initial and final beam cambers. The initial camber deflection multipliers are used to correct instantaneous camber calculations for differences between the minimum specified initial concrete compressive strength in the plans and the average initial compressive strength measured at the plant. (Camber is influenced directly by the concrete modulus of elasticity which is, in turn, computed from the concrete strength.) PPCBs with minimum specified initial strengths of 6 ksi or less will, on average, have significantly larger actual initial strengths. PPCBs with minimum specified initial strengths of more than 6 ksi have only moderately higher actual initial strengths. The final camber deflection multipliers are applied to the initial camber (i.e. modified instantaneous camber). The final multipliers account for the effects of creep which bring about camber growth. The magnitude of the initial camber determines which final camber deflection multiplier to apply. The final multipliers assume camber growth occurring over a period of 120 days with temporary blocking at L/30 from the beam ends where L is total beam length. Initial and final camber values are based on beams supported at their ends. BDM C5.4.1.4.1.9 contains a camber calculation example that closely approximates the method used in LBC for the determination of instantaneous camber and also illustrates the use of the camber deflection multipliers.

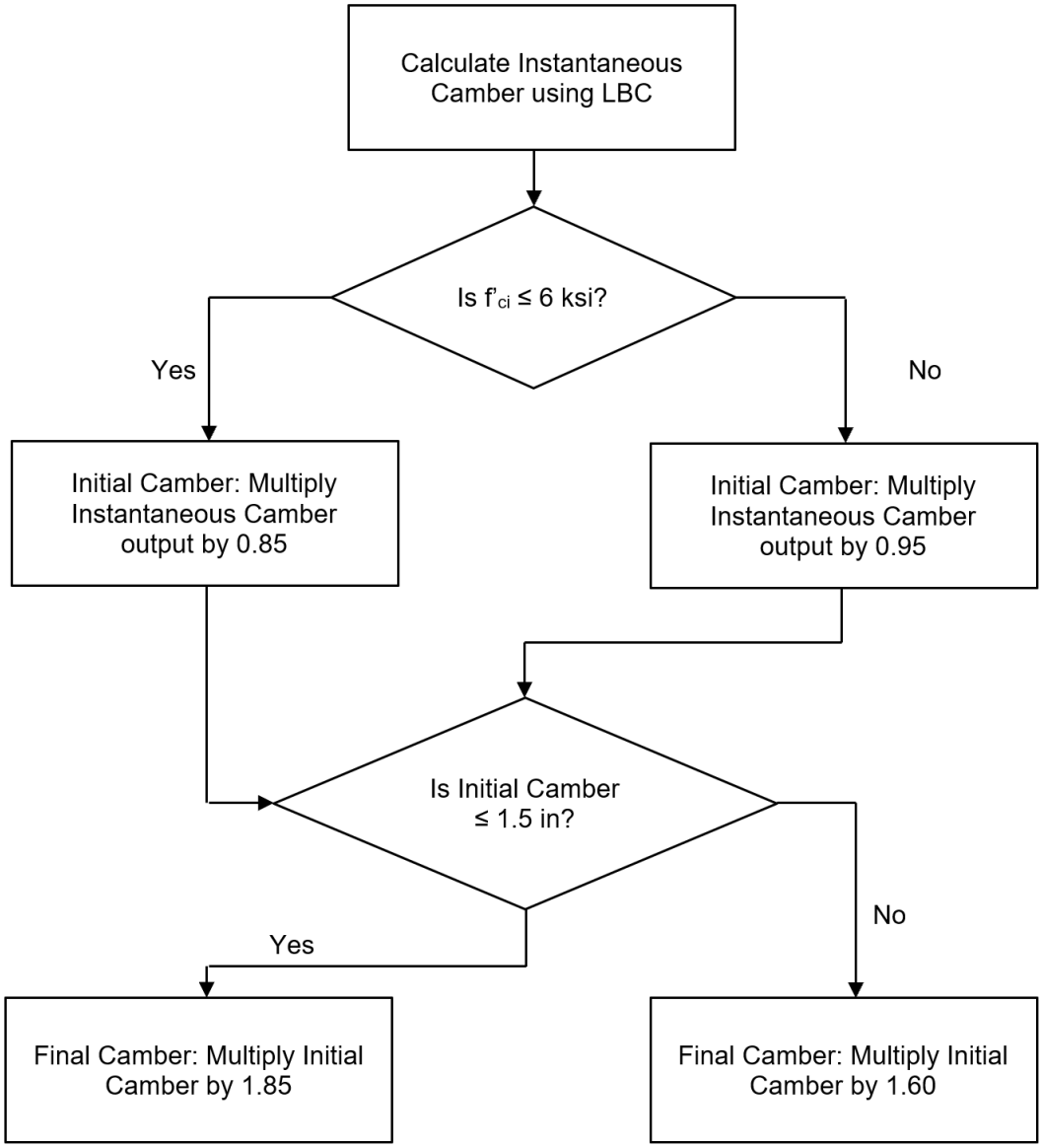


Figure 5.4.1.4.1.9. Initial and Final Camber Determination

Each standard sheet for a beam shape lists data for camber and deflection. The designer should note the loads used for the standard beam deflection computations and the continuity adjustments for long-term deflections. Long-term deflections are adjusted with multipliers as follows:

- 1 for simple spans,
- ¾ for end spans of a continuous bridge, and
- ½ for interior spans of a continuous bridge.

5.4.1.4.1.10 Anchorage zone

[AASHTO-LRFD 5.9.4.4]

Anchorage zone reinforcing shall comply with AASHTO LRFD Specifications for bursting resistance in pretensioned anchorage zones [AASHTO-LRFD 5.9.4.4.1]. The LBC software reports on anchorage zone (splitting resistance) requirements, but does not explicitly flag non-compliant designs, therefore, users need to carefully review that these requirements have been met. Prestress force in strands which are

debonded in the anchorage zone may be neglected. Prestress force in bonded straight and deflected strands in the anchorage zone shall be included.

5.4.1.4.1.11 Handling and shipping

Top flanges of beams shall be designed with longitudinal reinforcing bars to resist tension due to prestressing, lifting overhangs, and shipping overhangs. The beam design shall permit the fabricator to substitute prestressing strands for top flange reinforcing bars and shall be detailed on the plans. With the strand substitution, each 0.6-inch strand shall be tensioned to a maximum of 5 kips. Note that the substitution need not be on a one-to-one basis, as indicated on the standard bulb tee sheets.

For nonstandard beams the designer shall investigate the lateral stability of the beam with the maximum allowable lateral sweep of L/80 [CMB IM 570] when the beam is supported at the lifting points. An acceptable procedure is given in PCI's *Precast Prestressed Concrete Bridge Design Manual* and *Recommended Practice for Lateral Stability of Precast, Prestressed Concrete Bridge Girders* [BDM 5.4.1.1.5].

The designer shall determine the maximum permissible shipping overhang for the service limit state and note the overhang on the beam plan. For standard BTB-BTE beams the permissible overhang may be taken from Table 5.4.1.4.1.11-1.

Table 5.4.1.4.1.11-1. Permissible shipping overhangs for standard BTB-BTE beams

Beam	Maximum Permissible Overhang, feet	Temporary Bracing (1)	BSB Standard Sheet (3)
BTB30-BTB75	5% of beam length (2)	---	4750
BTB80-BTB85	5	---	4750
BTB90	7.5	---	4750
BTB95	11	---	4750
BTB100-BTB105	11.5	Yes	4765
BTC30-BTC75	5% of beam length (2)	---	4700
BTC80-BTC95	11	---	4700
BTC100-BTC110	12	Yes	4700
BTC115	14	Yes	4700
BTC120	14	Yes	4719s1
BTD50-BTD90	5% of beam length (2)	---	4730
BTD95	10	---	4730
BTD100	9	---	4730
BTD105	11	---	4730
BTD110-BTD115	11	Yes	4730
BTD120-BTD130	14	Yes	4730
BTD135	16	Yes	4748s1
BTE60-BTE90	5% of beam length (2)	---	4770s2
BTE95-BTE120	10	---	4770s2
BTE125-BTE135	14	Yes	4770s2
BTE140-BTE150	16	Yes	4770s2
BTE155	16	Yes	4790s1

Table notes:

(1) The contractor shall assure the lateral stability of the beams during handling, transporting, and erection by providing temporary bracing as needed.

(2) The Standard Specifications [IDOT SS 2407.03, K, 1] limit the overhang length to 5% of beam length unless the length is specifically given in contract documents.

(3) Beams shall achieve final 28-day concrete strength (f'_c) as specified on the standard beam sheets prior to shipping.

For shipping overhangs not listed in the tables the designer shall check service limit state stresses at the assumed support location and the hold-down point for the draped strands. The following conditions shall be used for the checks.

- At the support location, use the beam dead load plus 100% for dynamic load allowance.
- At the hold-down location, use the beam dead load without dynamic load allowance. Assume a 10-foot overhang at the opposite end of the beam.

For overhang checks the designer shall use the following stress limits (which generally are not stated in the AASHTO LRFD Specifications).

- Tension in reinforcing bars: $0.50f_y$
- Tension in prestressing strand: $0.70f'_s$
- Compression in concrete after losses: $0.60f'_c$
- Tension in concrete: maximum of $0.237(f'_c)^{1/2}$ ksi. Where calculated tension stress exceeds 0.200 or $0.0948(f'_c)^{1/2}$ ksi bonded reinforcing shall be provided to resist the total tension force in the concrete computed for an uncracked section.

5.4.1.4.1.12 Additional considerations

In Construction and Materials Bureau Instructional Memorandum “Precast and Prestressed Concrete Bridge Units” [CMB IM 570, Appendix D] beam fabricators are given an option for stressing and anchoring prestressing strands during cold weather. The option, however, seldom needs to be considered during design.

For special beams the Bureau may need to investigate permissible fabricating options after the project is let. Approvals for fabricating options are routed to the Chief Structural Engineer.

5.4.1.4.2 Detailing

[AASHTO-LRFD 5.9.4.1, 5.10.1, 5.14]

Bid item codes for the beams are consistent. For example, the code for a BTD 115 is “Beams, Pretensioned Prestressed Concrete, BTD115”.

When the designer alters a standard beam’s concrete strength, strand pattern, debonding requirements, or length, the designer shall designate the beam as special with an “S.” The designer also shall use a special bid item (e.g. “Beams, Pretensioned Prestressed Concrete, SBTD115”) and note the reason for the “S” designation in the bid item reference list. The special designation should not be used for beams that have modified stirrup lengths, but the designer shall provide a note regarding the altered stirrup lengths in the bid item reference list.

For bridges with skews greater than 30 degrees, the wide flanges of BTB-BTE beams may pose detailing problems. In those cases, the flanges may be chamfered more than the standard 2 inches. If it appears that extra chamfering or other special cutting is necessary, the designer shall consult with the supervising Unit Leader.

Except for unusually severe exposures, clear cover over stirrups shall be 1.0 inch minimum, and clear cover over prestressing strands and main reinforcing bars shall be 1.5 inch minimum. These amounts of cover follow past Bureau practice and generally are within the limits set by the AASHTO LRFD Specifications [AASHTO-LRFD 5.14.3 & 5.10.1].

Exceptions to the cover rules given above are the reduced cover at a one-half inch thick beam seat protection plate and at a 1.0-inch thick curved sole plate [BSB SS 4541]. For special beams the designer may reduce cover for a curved sole plate provided that the plate is not thicker than 1.0 inch.

Center-to-center spacing of 0.6 inch strands shall be a minimum of 2.0 inches [AASHTO-LRFD 5.9.4.1].

Lifting points and lifting loop details shall be included as shown on the standard beam sheets.

Shipping hold-down details consisting of two tie-down holes at each end of the beam may be cast into the web using sleeves up to 3 inches in diameter for BTD and BTE beams. The centerline of the holes shall be a minimum of 15 inches below the top flange of the beam and shall be 2 inches clear of any strands. The location of the tie down holes shall follow the beam overhang limits shown on the standard beam sheets. See CMB IM 570 Appendix J for more information. Tie-downs over the top of the top flanges on the taller BT beams have led to concrete damage at the flange edges during shipping. The shipping tie-down holes in the web avoid the issue.

All mild reinforcement in the standard beams has lap and development lengths based on requirements for epoxy-coated bars. The beam standards note that fabricators may elect to use epoxy-coated reinforcing for all mild reinforcement in the beams. At a minimum and as shown in the standards, the beam stirrups that extend into the deck must be epoxy-coated.

If deck hangers are embedded in PPCBs they shall be coated for protection [IDOT SS 2412.03, A, 2].

At integral abutments and at fixed piers with plain neoprene pad bearings, no special end bearing details are required to be cast in the beam ends. At expansion piers and fixed piers where the beam bears on a steel pintle plate or masonry plate, then curved sole plates must be cast in beam ends. Typical sole plate details are given on a standard sheet [BSB SS 4541].

Except where a curved sole plate is required, fabricators have permission to add a 1/2-inch thick steel beam seat protection plate with ¾-inch diameter welded studs in each end of a standard beam [CMB IM 570, Appendix H and H1]. The plate prevents damage to the beam end when strands are released and the beam cambers upward at transfer.

Coil ties to provide continuous transverse reinforcement are required at all abutments and at all pier diaphragms. See standard sheets for number and locations.

At a concrete end diaphragm some of the prestressing strands are to be cut with projections and shop-bent to hook into the cast-in-place diaphragm. Locations of the strands to be bent and dimensions of the hooks are given on standard beam detail sheets. All debonded strands shall be cutoff flush with the beam ends and shall not be embedded into the pier or abutment diaphragm.

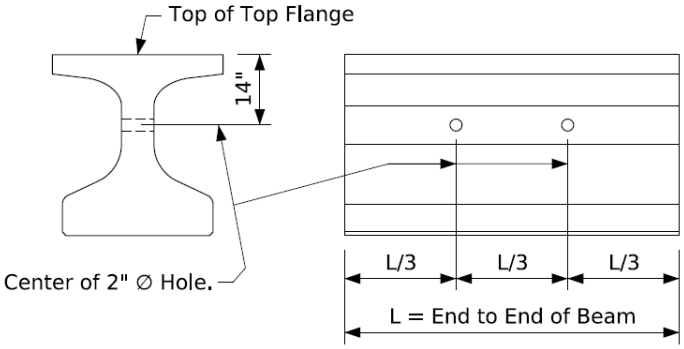
Seal the openings between debonded strands and sheathing before coating of the beam ends per the plans and CMB IM 570.

At expansion joints in PPCB superstructures, beam ends shall be sealed [CMB IM 570], and the designer shall note the requirement on the plans. The ends shall be sealed in the fabrication yard and again in the field when the beam seats are sealed [BDM 6.5.4.1.2]. BSB SS 4542-BTCDE to 4548-BTCDE detail the concrete sealer limits.

For all BTB-BTE standard beams with spans up to and including 120 feet use one steel diaphragm at midspan. For BTD and BTE standard beams with spans greater than 120 feet, two diaphragms are required at 20 feet on each side of the center of the beam. For skews of 7.5 degrees or less the diaphragms are to be skewed; for larger skews, diaphragms are to be perpendicular to the beams and staggered as shown on the standard sheets. At any location below a longitudinal bridge deck construction joint, the bolts for a steel intermediate diaphragm shall not be tightened until stage two of the bridge deck has been placed.

For bridges constructed with BTB-BTE standard beams, the deck overhang should be 3.50 feet from centerline of beam. The overhang allows about 2-inches clearance between a 4-inch deck drain and the edge of the bulb tee flange.

When preliminary design indicates that the design or lesser flood at a bridge site would extend above the bottom flanges of beams, during a flood the superstructure will be subjected to uplift from the trapped air between beams. In that case the preliminary design (or type, size, and location, commonly called the TS&L) will have a recommendation for venting of the superstructure. The designer shall provide vent holes in beam webs, 2 inches in diameter, about 14 inches below the top of the top flange, and at the third points of beam length, as shown on Figure 5.4.1.4.2. The holes may be provided with a removable or non-removable form. Locations of the holes may be shifted slightly to avoid interference with reinforcing.



NOTE: 2" ∅ holes to be provided in every prestressed beam.
2" ∅ holes may be produced with a removable or non-removable form.
They may be shifted slightly as needed to avoid interference with reinforcing, strands, and diaphragm connections.

Figure 5.4.1.4.2. Vent hole detail