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5.4 Pretensioned Prestressed Concrete Beams

This article now covers only the PPCB LRFD superstructure type. The recent series of beams are summarized in Table 5.4.

Table 5.4. Pretensioned prestressed concrete beam series and AASHTO design specifications

Beam Series	AASHTO Specification
BTB-BTE	LRFD, 4th Edition
A-D	LRFD, 4th Edition

5.4.1 PPCB LRFD [AASHTO-LRFD Section 5]

5.4.1.1 General

The standard A-D beams, the standard BTB-BTE beams, and procedures described in this article meet AASHTO LRFD Specifications, Fourth Edition, with minor modifications. 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]. At this time not all of the articles referenced above have been updated to the AASHTO LRFD Specifications.

5.4.1.1.1 Policy overview

The office-designed A-D standard beams and BTB-BTE standard bulb tee beams are available from approved fabricators. These standard beams are designed for simple span conditions and also 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].

Generally the A-D beams are preferred for cost and detailing reasons, but the BTB-BTE beams may be better choices depending on the following considerations:

- Spans are greater than 110 feet (33.500 m),
- Minimum vertical clearance is an issue,
- Profile grade adjustments need to be minimized,
- Skews are 30 degrees or less,
- Long spans, vertical curves, or horizontal curves do not cause excessive haunches, and
- Longer spans will reduce the number of piers.

These considerations are discussed in more detail in the preliminary design section [BDM 3.2.6.1.6].

The office assembles pretensioned prestressed concrete beam (PPCB) superstructures for short- to intermediate-span bridges from the standard beams and diaphragms. Each beam shape, A through D and 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 7.50 feet (2.286 m) for the A-D beams and 9.25 feet (2.820 m) 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 I and bulb tee pretensioned prestressed concrete beams designed for HL-93 loading

Beam Shape	OBS Standard Sheets		Span Range, feet (m)	Depth, inches (mm)	Maximum Spacing feet (m)
	Beams	Diaphragms			
A	4600-4602	1036, 1036A	30.00 to 55.00 (9.144 to 16.764)	32 (813)	7.50 (2.286)
B	4610-4612	1036, 1036A	34.17 to 67.50 (10.415 to 20.574)	39 (991)	7.50 (2.286)
C	4620-4623	1036, 1036A	30.00 to 80.00 (9.144 to 24.384)	45 (1143)	7.50 (2.286)
D	4630-4636	1036, 1036A	35.00 to 110.00 (10.668 to 33.528)	54 (1372)	7.50 (2.286)
BTB	4750-4767	1036-BTBR, 1036-BTBW (1)	30.00 to 105.00 (9.144 to 32.004)	36 (914)	9.25 (2.819) (2), (3)

BTC	4700-4719	1036CR, 1036CW (1)	30.00 to 120.00 (9.144 to 36.576)	45 (1143)	9.25 (2.819) (2), (3)
BTD	4730-4748	1036DR, 1036DW (1)	50.00 to 135.00 (15.240 to 41.148)	54 (1372)	9.25 (2.819) (2), (3)
BTE	4770-4790	1036-BTER, 1036-BTEW (1)	60.00 to 155.00 (18.288 to 47.244)	63 (1600)	9.25 (2.819) (2), (3)

Table notes:

(1) With the BTB-BTE shapes there is no concrete diaphragm option.
(2) All maximum spacings are for steel diaphragms only.
(3) Maximum spacing is reduced for the longest beams: BTB100, BTB105, BTC120, BTD135, and BTE155. See standard sheets for spacing limits.

Guidelines in this article [BDM 5.4.1] are provided to document the design criteria for the standard I and bulb tee beam shapes and for designing modified beams. When the designer changes the concrete strength, strand pattern, 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 Section Leader for guidance.

5.4.1.1.2 Design information

In the office, for individually designed bridges the preliminary design section selects the standard I or 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 I and bulb tee shapes are given on standard sheets [BDM Table 5.4.1.1.1]. It is office 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 Office of Materials Instructional Memorandum “Precast and Prestressed Concrete Bridge Units” [OM IM 570] and Appendices contain information on beam repair, cold weather placement, combined beam pours, and quality control. The memo is available from the Office of Materials and also 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.

Standard I refers to the A-D beam shapes.

Section Leader is the supervisor of the Office of Bridges and Structures preliminary bridge section, final design section, or consultant coordination section.

5.4.1.1.4 Abbreviations and notation [AASHTO-LRFD 3.3.2, 3.10.4.2]

A, B, C, and D, standard I-shapes
BTB, BTE, standard bulb tee beam shapes with depths of 36 inches (914 mm) and 63 inches (1600 mm), respectively [OBS SS 4740-4756 and 4760-4780]

BTC, BTD, standard bulb tee beam shapes with depths of 45 inches (1143 mm) and 54 inches (1372 mm), respectively [OBS SS 4700-4748]
CONSPAN (or 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].
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
L, span length center to center of bearings
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
$\mathbf{S_{D1}}$**,** 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-99, Standard Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete.” *2000 Annual Book of ASTM Standards*. West Conshohocken: ASTM, 2000.

Barker, R.M. and J.A. Puckett. *Design of Highway Bridges, An LRFD Approach, Second Edition.* New York: John Wiley and Sons, Inc., 2006.

“Concrete Superstructure US.pdf.” Washington, DC: American Association of State Highway and Transportation Officials (AASHTO). (A prestressed concrete bridge design example based on the second edition of the AASHTO LRFD specifications, which is available on the Internet at: http://tig.transportation.org/?siteid=34&pageid=339)

Gheitanbaf, E.H., J. Nervig, W. He, S. Sritharan, and J.M. Rouse. *Improving Accuracy of the Camber Predictions for Precast Prestressed Concrete Bridge Girders.* IHRB Project TR-xxx, December 2014.

Lin, T.Y. and Ned H. Burns. *Design of Prestressed Concrete Structures, Third Edition.* New York: John Wiley and Sons, 1981.

Nilson, Arthur H. *Design of Prestressed Concrete, Second Edition.* New York: John Wiley and Sons, 1987.

Office of Construction. *Construction Manual.* Ames: Office of Construction, Iowa Department of Transportation, 2006. (Available on the Internet at: http://www.iowadot.gov/erl/current/CM/Navigation/nav.pdf)

Office of Materials. *Precast and Prestressed Concrete Bridge Units, Materials Instructional Memo 570.* Ames: Iowa Department of Transportation, October 19, 2010. (Available on the Internet at: http://www.iowadot.gov/erl/current/IM/Navigation/index_number.htm)

Precast/Prestressed Concrete Institute (PCI). *Bridge Design Manual.* Chicago: PCI, 1997 with revisions through 2004. (Available on the Internet at: http://www.pci.org/publications/bridge/)

Precast/Prestressed Concrete Institute (PCI). *PCI Design Handbook, Sixth Edition.* Chicago: PCI, 2004.

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 office 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 slab between beams is simply supported and all of the deck weight of an overhang is distributed to the exterior girder.

For a typical PPCB bridge the office requires a future wearing surface load of 0.020 ksf (960 Pa), which is part of the DW load.

For two course decks the designer shall consider the second course as DC2 load because it will be placed under relatively controlled conditions by the bridge contractor.

For superstructures with roadway widths no greater than 44 feet (13.200 m), DC2 and DW shall be distributed equally to all beams.

For superstructures with roadway widths greater than 44 feet (13.200 m), 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].

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].

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 (3.60 kPa) 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 beams shall be as given in the AASHTO LRFD Specifications [AASHTO-LRFD 4.6.2.2], and the exterior beams shall have at least the same capacity. 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].

For determining the live load shear and moment distribution factors for an exterior beam the designer shall follow the guidelines below.

(1) For exterior beam design with the slab cantilever length (distance from the centerline of the exterior beam to the edge of the deck) equal to or less than one-half of the adjacent interior beam spacing, use the live load distribution factor for the interior beam.
(2) For exterior beam design with slab cantilever length exceeding one-half of the adjacent interior beam spacing, use the lever rule with the multiple presence factor of 1.0 for single lane to determine the live load distribution. The live load used to design the exterior girder shall never be less than the live load used to design an interior girder.
(3) The designer shall not use the analysis based on the conventional approximation of loads on piles [AASHTO-LRFD 4.6.2.2.2d].

If an exterior beam needs a greater load capacity than an interior beam the designer shall check with the supervising Section Leader.

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 I and bulb tee beams the designer will not need to check constructability but, if a check is required under the AASHTO LRFD Specifications [AASHTO-LRFD 6.10.3] because of unusual conditions, the office requires the designer to use the following loads.

- Dead load of forms: 0.010 ksf (480 Pa)
- Dead load of edge rail and walkway applied at the edge of the deck form: 0.075 klf (1100 N/m)
- Construction live load: 0.050 ksf (2400 Pa) [IDOT SS 2403.03, L, 4, d.]
- Live load of finishing machine located along the edge of the deck form to maximize the design condition: 6 kips (26.690 kN)
- Wind load for 100 mph (161 kph) wind at elevations not exceeding 30 feet (9.140 m) above ground level: 0.050 ksf (2400 Pa). At higher elevations the load shall be adjusted upward.

The designer shall consider the deck pouring sequence in the design of the girders for constructability.

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.

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.

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, 5.9.3, 5.9.4.1, 5.9.4.2]

The standard beams have been checked for compression and tension stresses at transfer (before losses) [AASHTO-LRFD 5.9.4.1]. 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 [AASHTO-LRFD 3.4.1].

- Strength I, superstructure with vehicles but without wind
- Extreme Event II, collision by vehicle (railing)
- Service I, superstructure with vehicles (Although the Service I limit state includes wind at 55 mph (89 kph), 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
- Fatigue

Usually Strength III (superstructure without vehicles but with wind exceeding 55 mph (89 kph)), Strength IV (superstructure with very high dead load to live load ratios), and Strength V (superstructure with vehicles and with wind at 55 mph (89 kph)) 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 strength limit states during construction. The construction limit states should be considered with respect to deck pour sequence, deck overhang brackets, and other applicable factors [AASHTO LRFD 2.5.3].

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, 5.9.3, 5.9.4.2].

5.4.1.4 A-D and BTB-BTE beams

Analysis, design, and detailing information in this article is provided primarily to document the conditions under which the standard I and 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 5.11.4.1]

For service and strength limit states, a standard I or 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 office 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 A-D and BTB-BTE beams and loads to the substructure

Load	Theoretical Span and Composite Conditions	Office 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 (13.200 m). 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) Office policy is to design beams using the simple span condition for all strength and service stress checks and add longitudinal slab 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 5.14.1.4.1, C5.14.1.4.1]. 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 end of a pretensioned beam [AASHTO-LRFD 5.11.4.1]. For the usual 0.6-inch (15.24-mm) diameter strand, 60 diameters is a distance of 36 inches (914 mm).

For design of A-D and BTB-BTE beams the deck haunch is assumed to be 1.00 inch (25 mm) and 1.50 inches (40 mm), respectively for weight and 0.50 inch (13 mm) 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 1.50 feet (500 mm) for the standard A-D beams and 1.92 feet (584 mm) 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 and 5.4.1.4.1.2-2.

Table 5.4.1.4.1.2-1. Concrete specifications for standard A-D pretensioned prestressed concrete beams

Beam Type	OBS Standard Sheet	f'_{ci}, ksi (MPa) (1)	f'_c, ksi (MPa) (2)
A30-A42	4600	4.5 (31.0) (3)	5.0 (35)
A46-A55	4600	6.0 (41.4)	7.0 (48.3)
B34-B50	4610	4.5 (31.0) (3)	5.0 (35)
B55-B67	4610	6.0 (41.4)	7.0 (48.3)
C30-C67	4620	4.5 (31.0) (3)	5.0 (35)
C71-C80	4620	5.0 (34.5)	6.0 (41.4)
D35-D95	4630	4.5 (31.0) (3)	5.0 (35)
D100-D105	4630	6.0 (41.4)	7.5 (51.7)
D110	4636	6.5 (44.8)	7.5 (51.7)

Table notes:

(1) For modified beams, the designer may increase the maximum f'_{ci} to 7.0 ksi (48.3 MPa). With approval of the supervising Section Leader, the maximum f'_{ci} may be increased to 8.0 ksi (55.2 MPa).

(2) For modified beams, maximum f'_c shall be 8.5 ksi (58.6 MPa). With approval of the supervising Section Leader, the maximum f'_c may be increased to 9.0 ksi (62.1 MPa).

(3) Strength is the value given in the standard specifications [IDOT SS 2407.03, B, 3].

In all cases the usual Class C concrete deck placed on the beams has f'_c equal to 4.0 ksi (28 MPa).

Table 5.4.1.4.1.2-2. Concrete specifications for standard BTB-BTE pretensioned prestressed concrete beams

Beam Type	OBS Standard Sheet	f'_{ci}, ksi (MPa) (1)	f'_c, ksi (MPa)	
			Beam (2)	Deck
BTB30-BTB65	4750	4.5 (31.0)	5.0 (34.5)	4.0 (28)
BTB70	4750	5.0 (34.5)	5.5 (37.9)	4.0 (28)
BTB75	4750	5.5 (37.9)	6.5 (44.8)	4.0 (28)
BTB80	4750	6.0 (41.4)	7.0 (48.3)	4.0 (28)
BTB85	4750	6.5 (44.8)	7.5 (51.7)	4.0 (28)
BTB90	4750	7.5 (51.7)	8.5 (58.6)	4.0 (28)
BTB95	4750	8.0 (55.2)	9.0 (62.1)	4.0 (28)
BTB100-BTB105	4765	8.0 (55.2)	9.0 (62.1)	4.0 (28)
BTC30-BTC60	4700	4.5 (31.0)	5.0 (34.5)	4.0 (28)
BTC65-BTC75	4700	5.0 (34.5)	6.0 (41.4)	4.0 (28)
BTC80-BTC90	4700	6.0 (41.4)	7.0 (48.3)	4.0 (28)
BTC95	4700	6.5 (44.8)	7.5 (51.7)	4.0 (28)
BTC100	4700	7.0 (48.3)	8.0 (55.2)	4.0 (28)
BTC105	4700	7.5 (51.7)	8.5 (58.6)	4.0 (28)
BTC110-BTC115	4700	8.0 (55.2)	9.0 (62.1)	4.0 (28)
BTC120	4719	8.0 (55.2)	9.0 (62.1)	4.0 (28)
BTD50-BTD70	4730	4.5 (31.0)	5.0 (34.5)	4.0 (28)
BTD75-BTD85	4730	5.0 (34.5)	6.0 (41.4)	4.0 (28)
BTD90-BTD95	4730	5.5 (37.9)	6.5 (44.8)	4.0 (28)
BTD100-BTD105	4730	6.0 (41.4)	7.0 (48.3)	4.0 (28)
BTD110	4730	6.5 (44.8)	7.5 (51.7)	4.0 (28)
BTD115	4730	7.0 (48.3)	8.0 (55.2)	4.0 (28)
BTD120	4730	7.0 (48.3)	8.0 (55.2)	4.0 (28)
BTD125	4730	7.5 (51.7)	8.5 (58.6)	4.0 (28)
BTD130	4730	8.0 (55.2)	9.0 (62.1)	4.0 (28)
BTD135	4748	8.0 (55.2)	9.0 (62.1)	4.0 (28)
BTE60-BTE95	4770 (Sheet 1 of 2)	4.5 (31.0)	5.0 (34.5)	4.0 (28)
BTE100-BTE110	4770 (Sheet 1 of 2)	5.0 (34.5)	5.5 (37.9)	4.0 (28)
BTE115-BTE120	4770 (Sheet 1 of 2)	5.5 (37.9)	6.0 (41.4)	4.0 (28)
BTE125	4770 (Sheet 1 of 2)	6.0 (41.4)	7.0 (48.3)	4.0 (28)
BTE130	4770 (Sheet 1 of 2)	6.5 (44.8)	7.5 (51.7)	4.0 (28)
BTE135	4770 (Sheet 1 of 2)	7.0 (48.3)	8.0 (55.2)	4.0 (28)
BTE140	4770 (Sheet 1 of 2)	7.5 (51.7)	8.5 (58.6)	4.0 (28)
BTE145	4770 (Sheet 1 of 2)	7.5 (51.7)	8.5 (58.6)	4.0 (28)
BTE150	4770 (Sheet 1 of 2)	8.0 (55.2)	9.0 (62.1)	4.0 (28)
BTE155	4790 (Sheet 1 of 2)	8.0 (55.2)	9.0 (62.1)	4.0 (28)

Table notes:

(1) For modified beams, the designer may increase the maximum f'_{ci} to 7.0 ksi (48.3 MPa). With approval of the supervising Section Leader, the maximum f'_{ci} may be increased to 8.0 ksi (55.2 MPa).

(2) For modified beams, maximum f'_c shall be 8.5 ksi (58.6 MPa). With approval of the supervising Section Leader, the maximum f'_c may be increased to 9.0 ksi (62.1 MPa).

The modulus of elasticity for pretensioned prestressed concrete beams shall be based on a concrete weight of 0.150 kcf (2400 kg/m^3). This weight is larger than given in the AASHTO LRFD Specifications [AASHTO-LRFD C5.4.1.4] 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 (28 MPa). Reinforcing steel shall be Grade 60, with a minimum yield strength of 60 ksi (400 MPa). Modulus of elasticity for reinforcing shall be taken as 29,000 ksi (200,000 MPa) [AASHTO-LRFD 5.4.3.2].

Prestressing strands shall be ASTM A416 (A416M) Grade 270 (1860) 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 (mm)	Nominal Area, in^2 (mm^2)	Ultimate Tensile Strength, kips (kN)	Maximum Initial Prestress (1), kips (kN)	Modulus of Elasticity (2), ksi (MPa)
½ (12.70) (3)	0.153 (98.7)	41.300 (183.7)	---	28,500(196,000)
0.6 (15.24)	0.217 (140.0)	58.600 (260.7)	43.950 (195.5)	28,500(196,000)

Table notes:

(1) The maximum initial prestress value in the table for design is 75% of guaranteed ultimate tensile strength (GUTS). The office 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 office 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 [OM IM 570, Appendix C].

(3) These ½-inch (12.70-mm) strands are to be used for lifting loops only. All stressed prestressing strands in the standard beams shall be 0.6 inches (15.24 mm) in diameter.

5.4.1.4.1.3 Design resistance and stress limits [AASHTO-LRFD 5.9.4.1, 5.9.4.2]

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

Stress limits for beams at transfer shall be as follows [AASHTO-LRFD 5.9.4.1].

- Compression: $0.60f'_{ci}$
- Tension: See the AASHTO LRFD table [AASHTO-LRFD Table 5.9.4.1.2-1].

Stress limits during service limit states shall be as follows [AASHTO-LRFD 5.9.4.2].

- Compression, Service Limit State Load Combination I: See the AASHTO LRFD table [AASHTO-LRFD Table 5.9.4.2.1-1].
- Tension, Service Limit State Load Combination III: See the AASHTO LRFD table [AASHTO-LRFD Table 5.9.4.2.2-1].

5.4.1.4.1.4 Section properties [AASHTO-LRFD 4.6.2.6.1, 5.7.1]

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. Strands should be transformed to PPCB concrete using the modular ratio for each steel material. The modular ratio should be rounded to the nearest whole number in accordance with the AASHTO LRFD Specifications [AASHTO-LRFD 5.7.1].

For estimating camber using LEAP CONSPAN, the designer shall use the transformed section option. Do not transform mild longitudinal bars or strands stressed to not more than 5 kips (22 kN) which are substituted for mild longitudinal bars in the top flange. CONSPAN does not round the strand to concrete modular ratio to the nearest whole number when determining the transformed section properties.

CONSPAN 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 0.5 inch (13 mm) 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 unrounded 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

Deflected 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 generally need to be used only in longer beams, and the number of deflected strands generally should be limited to 25 to 33% of the total number of strands.

Deflected strands also increase the shear strength near beam ends, and the deflected strands should be considered when determining shear reinforcement.

Deflected strands in the standard I and bulb tee beams are shown on standard sheets [OBS SS 4601-4602, 4611-4612, 4623, 4633-4636, 4757-4764, 4766, 4768 4708-4718, 4719s2, 4733-4747, 4748s2, 4777-4789, 4790s2] 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 ± 1 inch (25 mm). The designer shall investigate beam stresses for the strand group at each of the extreme tolerances.

5.4.1.4.1.6 Prestress losses [AASHTO-LRFD 5.9.5]

Prestress losses shall be determined in accordance with specific AASHTO provisions for elastic shortening, creep, shrinkage, and relaxation [AASHTO-LRFD 5.9.5]. The designer shall use the approximate method for the time-dependent losses [AASHTO-LRFD 5.9.5.3].

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 5.7.3.3]

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, and
- Midspan.

The maximum and minimum amounts of prestressing and reinforcing steel in beams shall meet the limits in the AASHTO LRFD Specifications [AASHTO-LRFD 5.7.3.3].

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 [OBS SS 4380-4385, 4380-BTB-4 – 4385-BTE-6, 4556-4561, 4556-BTC-4 – 4561-BTE-6] have the correct negative moment reinforcing for the span lengths available with standard PPCBs. 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 (#29). 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 office 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.

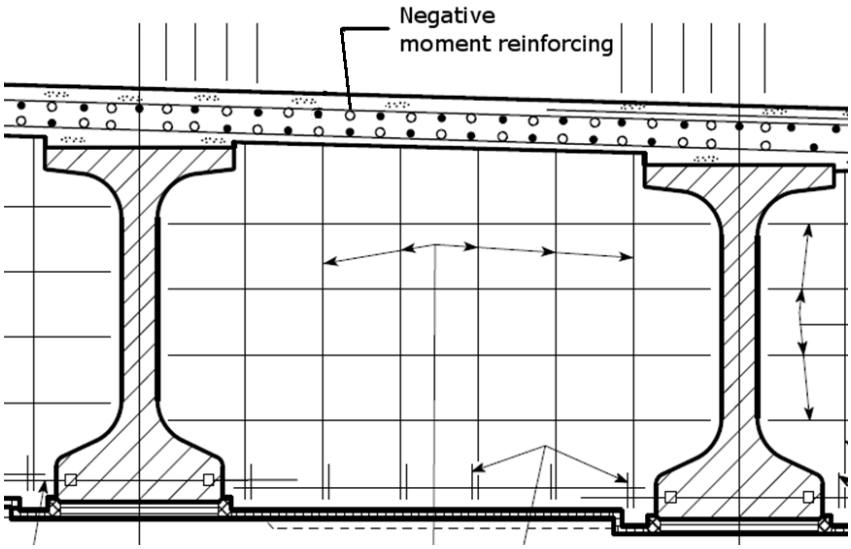


Figure 5.4.1.4.1.7. Negative moment reinforcement above piers

5.4.1.4.1.8 Shear [AASHTO-LRFD 5.8.3, 5.8.4]

Web shear reinforcement shall be designed in accordance with AASHTO LRFD Specifications [AASHTO-LRFD 5.8.3]. The vertical component of the deflected strand prestressing force should be included, if present, when calculating beam shear strength.

For a standard beam, office 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 5.8.4]. 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 (6 mm) except for a 2-inch (50-mm) wide finish along one edge of the flange [OM IM 570]. The strip with the smooth finish is intended to aid in taking elevations during construction.

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 A-D beams meet this criterion, BTB beams meet this criterion to 91 feet (27.940 m), BTC beams meet this criterion to 110 feet (33.655 m), BTD beams meet this criterion to 129 feet (39.370 m), and BTE beams meet this criterion to 147 feet (45.085 m) [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 Section Leader.

The office 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 (3.600-meter) 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 Section 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 will be incorporated into the PPCB standard sheets in early 2015.

The flowchart in Figure 5.4.1.4.1.9 illustrates the camber deflection multipliers applied to the theoretical instantaneous camber calculation from CONSPAN 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 (41 MPa) or less will, on average, have significantly larger actual initial strengths. PPCBs with minimum specified initial strengths of more than 6 ksi (41 MPa) 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 CONSPAN for the determination of instantaneous camber and also illustrates the use of the camber deflection multipliers.

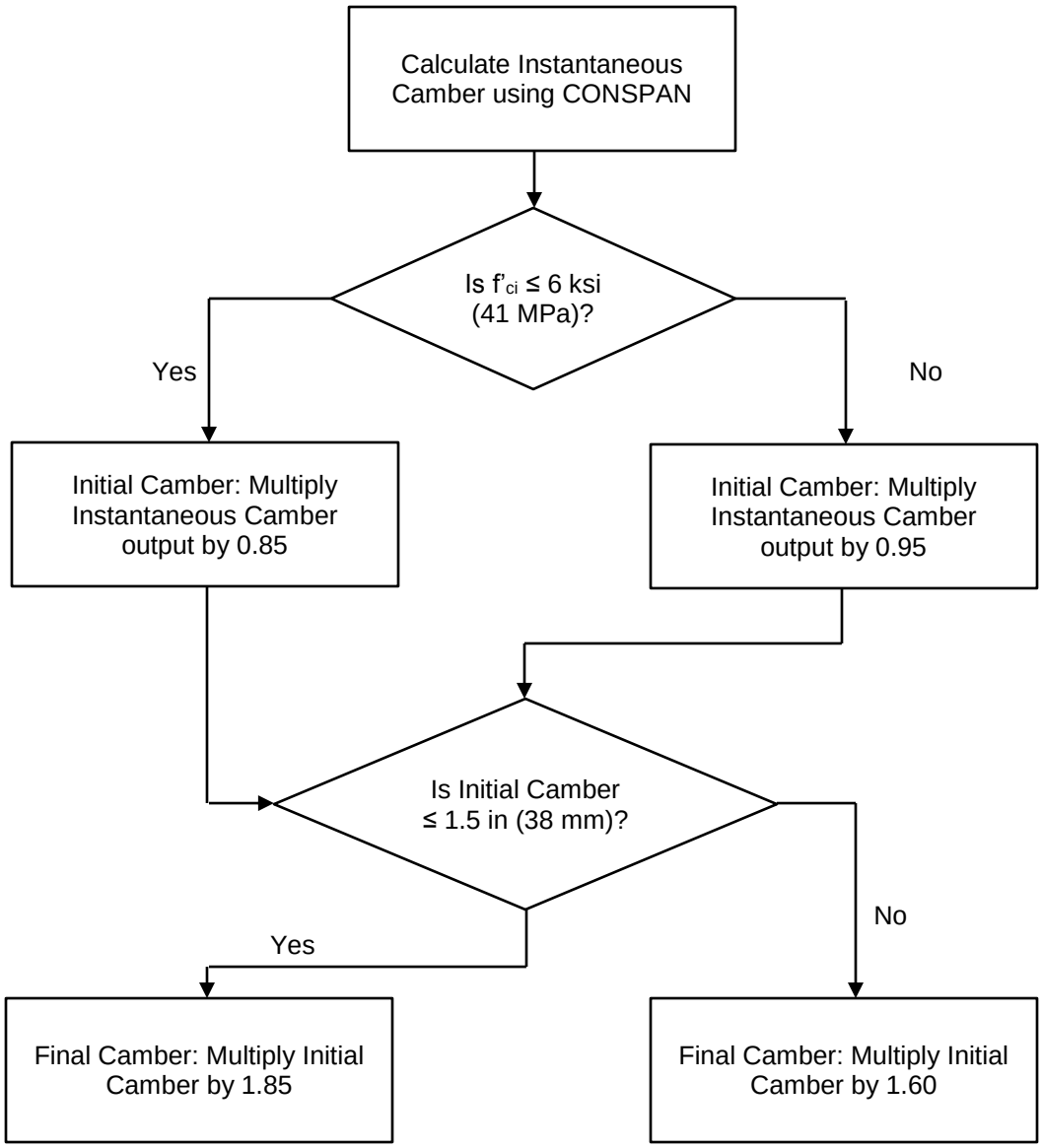


Figure 5.4.1.4.1.9. Initial and Final Camber Determination

Each standard sheet for a beam shape lists data for camber and deflection [OBS SS 4600, 4610, 4620, 4630, 4636, 4750, 4765, 4700, 4719s1, 4730, 4748s1, 4770s1, 4790s1]. 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 5.10.10]

Anchorage zone reinforcing shall comply with AASHTO LRFD Specifications for bursting resistance in pretensioned anchorage zones [AASHTO-LRFD 5.10.10.1].

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 (15.24-mm) strand shall be tensioned to a maximum of 5 kips (20 kN). Note that the substitution need not be on a one-to-one basis, as indicated on the standard bulb tee sheets [OBS SS 4751-4768, 4701-4719s2, 4731-4748s2, 4770s1-4790s2].

For nonstandard beams the designer shall investigate the lateral stability of the beam with the maximum allowable lateral sweep of L/80 [OM IM 570] when the beam is supported at the lifting points. An acceptable procedure is given in PCI's *Bridge Design Manual* [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 D and BTB-BTE beams the permissible overhang may be taken from Table 5.4.1.4.1.11-1 and Table 5.4.1.4.1.11-2.

Table 5.4.1.4.1.11-1. Permissible shipping overhangs for standard D beams

Beam	**Maximum Permissible Overhang, feet (m)**		**Temporary Bracing(1)**	**OBS Standard Sheet**
	Minimum f'_c at Shipping			
	5.0 ksi (35 MPa)	**7.5 ksi (52 MPa)**		
D35-D80	5% of beam length(2)		---	4630
D85	8 (2.438)		---	4630
D90	9 (2.743)		---	4630
D95	11 (3.353)		---	4630
D100	---(3)	12 (3.658)	Yes	4630
D105	---(3)	12 (3.658)	Yes	4630
D110	---(3)	14 (4.267)	Yes	4636

Table notes:

(1) Where noted in the table for the longer beams, 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) Shaded cells represent concrete strengths that are not permissible for the standard beams.

Table 5.4.1.4.1.11-2. Permissible shipping overhangs for standard BTB-BTE beams

Beam	Maximum Permissible Overhang, feet (m)	Temporary Bracing (1)	OBS Standard Sheet
BTB30-BTB75	5% of beam length (2)	---	4750
BTB80-BTB95	11 (3.353)	---	4750
BTB100-BTB105	12 (3.658)	Yes	4765s1
BTC30-BTC75	5% of beam length (2)	---	4700
BTC80-BTC95	11 (3.353)	---	4700
BTC100-BTC110	12 (3.658)	Yes	4700
BTC115	14 (4.267)	Yes	4700
BTC120	14 (4.267)	Yes	4719s1
BTD50-BTD90)	5% of beam length (2)	---	4730
BTD95	11 (3.353)	---	4730
BTD100	10 (3.048)	---	4730
BTD105	12 (3.658)	---	4730
BTD110-BTD115	12 (3.658)	Yes	4730
BTD120	14 (4.267)	Yes	4730
BTD125-BTD130	16 (4.877)	Yes	4730
BTD135	16 (4.877)	Yes	4748s1
BTE60-BTE85	5% of beam length (2)	---	4770s2
BTE90-BTE100	11 (3.353)	---	4770s2
BTE105-BTE120	12 (3.658)	---	4770s2
BTE125-BTE135	14 (4.267)	Yes	4770s2
BTE140-BTE150	16 (4.877)	Yes	4770s2
BTE155	16 (4.877)	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.

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 impact.
- At the hold-down location, use the beam dead load without impact. Assume a 10-foot (3-m) 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 ($0.623(f'_c)^{1/2}$ MPa). Where calculated tension stress exceeds 0.200 or $0.0948(f'_c)^{1/2}$ ksi (1.4 or $0.249(f'_c)^{1/2}$ MPa) 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 Office of Materials Instructional Memorandum “Precast and Prestressed Concrete Bridge Units” [OM 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 office 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 5.10.3.3.1, 5.12]

Bid item codes for the bulb tee beams are consistent with codes for other standard beams. 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, or length, the designer shall designate the beam as special with an “S.” The designer also shall use a special bid item 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 (50 mm). If it appears that extra chamfering or other special cutting is necessary the designer shall consult with the supervising Section Leader.

Except for unusually severe exposures, clear cover over stirrups shall be 1.0 inch (25 mm) minimum, and clear cover over prestressing strands and main reinforcing bars shall be 1.5 inch (38 mm) minimum. These amounts of cover follow past office practice and generally are within the limits set by the AASHTO LRFD Specifications [AASHTO-LRFD 5.12].

Exceptions to the cover rules given above are the reduced cover at a one-half inch (13 mm) thick beam seat protection plate and at a 1.0-inch (25-mm) thick curved sole plate [OBS SS 4541-4541E]. For special beams the designer may reduce cover for a curved sole plate provided that the plate is not thicker than 1.0 inch (25 mm).

Center-to-center spacing of 0.6 inch strands shall be a minimum of 2.0 inches (50 mm) [AASHTO-LRFD 5.10.3.3.1].

Lifting points and lifting loop details shall be shown on the beam plan [OBS SS 4600, 4610, 4620, 4630, 4636, 4750, 4765, 4700, 4719s1, 4730, 4748s1, 4770s2, 4790s1].

All barrier rail to bridge deck/wing reinforcement for interstate and primary bridges shall be stainless steel. The following reinforcement in PPCB superstructures shall be epoxy coated:

- All deck reinforcing steel, longitudinal and transverse, top and bottom.
- All reinforcing in concrete diaphragms.
- All barrier rail and median barrier reinforcing steel, longitudinal and vertical, except as noted above.
- All light pole base reinforcing steel.
- All beam stirrups that extend into the deck.

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. At expansion piers and fixed piers with masonry plate bearings, curved sole plates must be cast in beam ends. Typical sole plate details are given on a standard sheet [OBS SS 4541A].

Except where a curved sole plate is required, fabricators have permission to add a 1/2-inch (12-mm) thick steel beam seat protection plate with ¾-inch (19-mm) diameter welded studs in each end of a standard beam [OM 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 [OBS SS 4600, 4610, 4620, 4630, 4750, 4765, 4700, 4719s1, 4730, 4748s1, 4770s2, 4790s1].

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 [OBS SS 4600, 4610, 4620, 4630, 4750, 4765, 4700, 4719s1, 4730, 4748s1, 4770s2, 4790s1].

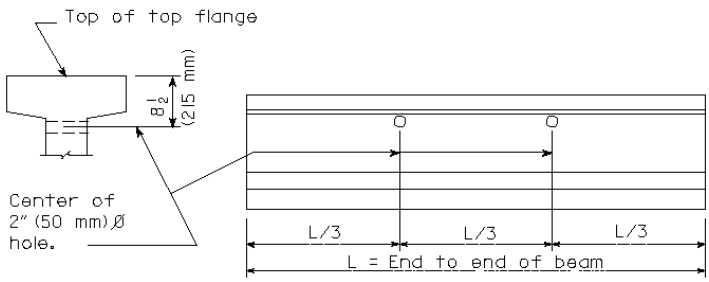
At expansion joints in PPCB superstructures, beam ends shall be sealed [OM IM 570], and the designer shall note the requirement on the plans. The ends shall be sealed in the fabrication yard and again in the field when the beam seats are sealed [BDM 6.5.4.1.2].

For A-D standard beams in superstructures above vehicular roadways, cast-in-place intermediate diaphragms are required [OBS SS 1036A]. For superstructures above railways or waterways the designer may select intermediate diaphragms of steel or cast-in-place concrete [OBS SS 1036]. For either choice one diaphragm at midspan is required. Coil ties are required at the locations of concrete diaphragms, and cast bolt holes are required at locations of steel diaphragms. At any location below a longitudinal bridge deck construction joint, a cast-in-place concrete intermediate diaphragm shall be omitted [OBS SS 1036A] and the bolts for a steel intermediate diaphragm shall not be tightened until stage two of the bridge deck has been placed [OBS SS 1036].

For BTB-BTE standard beams, only steel diaphragms are permitted, and the office standard diaphragm configuration varies depending on whether the bridge crosses a roadway or a waterway [OBS SS 1036-BTBR, 1036-BTBW, 1036CR, 1036CW, 1036DR, 1036 DW, 1036-BTER, 1036-BTEW]. For all spans up to and including 120 feet (36.580 m) one diaphragm at midspan is required. For BTD and BTE beams with spans greater than 120 feet (36.580 m) two diaphragms are required at 20 feet (6.100 m) 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.

For bridges constructed with BTB-BTE standard beams, the deck overhang should be 3.50 feet (1.067 m) from centerline of beam. The overhang allows about 2 inches (50 mm) clearance between a 4-inch (102-mm) 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 (50 mm) in diameter, about 8.5 inches (215 mm) 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" (50 mm)Ø Holes to be provided in every prestressed beam. 2" (50 mm)Ø holes may be produced with a removable or non-removable form. They may be shifted slightly to avoid interference with reinforcing.

Figure 5.4.1.4.2. Vent hole detail