SECTION 5
CONCRETE STRUCTURES

5.1 GENERAL REQUIREMENTS

The provisions in this section apply to the design of reinforced concrete and prestressed concrete.

5.2 CODE REQUIREMENTS

Designs shall be consistent with AASHTO, unless modified herein.

5.3 MATERIAL PROPERTIES

5.3.1 Concrete Classes

5.3.1.1 Cast-in-Place Concrete

Table 5-1 and Table 5-2 show CDOT's most commonly specified classes of cast-in-place (CIP) concrete, typical design 28-day compressive strengths, and typical uses. See CDOT Standard Specifications for more information on concrete classes. Class DF shall be used to replace Class D concrete on elements exposed to de-icing salts (Splash Zone) in new bridge structures. Do not substitute Class BZ or S with Class DF. If stainless reinforcing is used, the Class DF requirement may be waived. Class DF is not required for prestressed elements or precast wall panels since the element is generally in compression and will limit crack size and chloride intrusion. When substructure elements are in separate construction pours such as columns and pier caps, the use of Class DF for those elements outside of the splash zone are not required, i.e. the quantities for the column could be Class DF while the quantities for the pier cap could be shown as Class D. Class DF should be considered for elements that are in areas that are exposed to leakage such as behind or near inlets.

Table 5-1: Common Concrete Classes and Strengths

Concrete Class	D, DF, DR	BZ	S35	S40	S50	Shotcrete
f'c (ksi)	4.5	4	5	5.8	7.25	4.5

Table 5-2: Typical CIP Concrete Applications

Structural Element	Typical Concrete Class
CIP Reinforced Concrete	D or DF
CIP Post-Tensioned Concrete	D or DF, S35, or S40
Drilled Shafts	BZ
Spliced Girder Bridge Closure Pours	D or DF, S35, S40, or S50*
Initial Facing for Soil Nail Walls and Top-Down Caisson Walls	Shotcrete
Concrete Patching	DR

*It is CDOT's preference to avoid designs using Class S50 concrete due to past difficulty in meeting the required cracking tendency test. In cases where the supplier is known during design, S50 concrete may be evaluated for feasibility.

5.3.1.2 Precast Concrete

Shop produced precast concrete girders shall be Class PS concrete and shall be limited to the following maximum design strengths:

- f'ci = 6.5 ksi
- f'c = 8.5 ksi

Plans shall show minimum strengths required to meet design requirements. These design strengths shall be used for all strength and service design checks.

Higher design values of f'c and f'ci may be permitted for special cases, after conferring with local precast suppliers and with approval from Unit Leader in coordination with the State Bridge Engineer.

Class DC concrete is a dry cast method of concrete used for precast box culverts.

5.3.1.3 Lightweight Concrete

It is CDOT's preference to avoid the use of lightweight concrete due to difficulty in passing aggregate tests and associated concerns regarding freeze-thaw durability. However, when the supplier is known during design, lightweight concrete is permitted for use provided a suitable mix passing ASTM C66 and C672 requirements is submitted for approval by the supplier to CDOT Materials. Approval for the use of lightweight concrete by the Unit Leader in coordination with the concrete SMEs is contingent on the passing mix design. The rationale for using lightweight concrete shall be documented in the Structure Selection Report.

5.3.2 Modulus of Elasticity

The unreinforced concrete unit weight for use in calculating the modulus of elasticity shall be per AASHTO Table 3.5.1-1 and C5.4.2.4. ***AASHTO 5.4.2.4***

5.3.3 Relative Humidity

When calculating creep and shrinkage coefficients, relative ambient humidity shall be taken as 55 percent. ***AASHTO 5.4.2.3***

5.3.4 Reinforcement

5.3.4.1 Mild Steel

Mild steel should typically be designed with a yield strength of 60 ksi. However, the use of 75 ksi rebar is allowed to assist in meeting the seismic transverse reinforcement detailing requirements when required in Seismic Zone 1 (see Section 5.4.9 for more information).

Use of epoxy-coated mild steel is the standard of practice where corrosion resistant reinforcement is required per Section 5.4.5, but alternates such as stainless steel should be considered per Section 5.3.4.3.

5.3.4.2 Welded Wire Fabric

Reinforcement for CIP concrete should generally be detailed as rebar in the contract plans, except for shotcrete wall facing where it is typically advantageous to specify welded wire fabric (WWF). In other structure elements where WWF may be an economical substitution, it may be noted as an allowable substitution at the Contractor's option.

5.3.4.3 Stainless Steel and Corrosion Resistant Alloy Steel (CRAS)

Both stainless steel and CRAS are acceptable alternatives to epoxy-coated mild steel. When the Designer elects to use either of these for a project, it shall be documented in the Structure Selection Report. The Designer is responsible for determining appropriate lap lengths.

5.3.4.4 Glass FRP Rebar

Glass FRP rebar shall not be used unless approved by Unit Leader.

5.3.4.5 Epoxy Anchored Systems

Expansion type concrete anchors are undesirable because of the vibration and pullout concerns. Instead, drilled-in-place anchor bolts bonded to the supporting concrete with an approved two-part epoxy system may be used. Two-part epoxy systems shall be approved by Concrete SME and CDOT Materials.

If the anchor is in continuous tension, the Designer shall use only an epoxy system if it is approved for use in continual tension loading. Project approval will be by the Unit Leader in coordination with the concrete SMEs and CDOT Materials using NTPEP, APL or project specific material submittals. Many epoxy systems are not allowed if the anchor is in continuous tension. Refer to ACI 318 and ACI 355.4 for more information on use of post-installed adhesive anchors.

5.3.5 Prestressing Strand and Bars

Prestressing strand shall be 0.60 in. diameter, low-relaxation strand, with a design ultimate tensile strength of 270 ksi. One exception to this requirement is for precast panel deck forms for which strands shall be no larger than 3/8 in. diameter. Prestressing bars shall have a design ultimate tensile strength of 150 ksi. ***AASHTO 5.4.4***

Post Tensioning Institute does not permit the use of tensioned galvanized bars because during curing the zinc layer may react with the alkaline grout and may generate hydrogen. Hydrogen can reduce the ductility of steel bars. Effective long-term corrosion protection is provided by grouting uncoated bars inside plastic ducts. The alkaline cement grout passivates the bar surface and the plastic duct acts as a moisture barrier. Such corrosion protection requires special anchorage details to maintain threadability and corrosion protection. CDOT has adopted this policy.

5.3.6 Concrete Inserts

Material of concrete inserts/embeds that will be part of the permanent structure shall match the material used for the attachments (e.g., bolts). Dissimilar materials shall be avoided to prevent corrosion issues. Galvanized or stainless steel inserts are preferred.

5.4 REINFORCED CONCRETE

5.4.1 Bar Size Availability

Reinforcing bars larger than #11 (that is, #14 and #18) may be used to eliminate reinforcement congestion if availability from suppliers is verified through the Engineering Estimates and Market Analysis Unit.

5.4.2 Development and Splice Lengths

Development lengths shall be calculated per AASHTO. ***AASHTO 5.10.8***

The general notes sheet of the bridge plans shall no longer contain lap splice tables. The following tables are provided for Designer use in selecting lap splices for epoxy coated bars in slabs, walls, and footings, or other non-stirrup contained reinforcing.

Table 5-3: Minimum Lap Length for Epoxy-Coated Slab, Wall, or Footing Bars Spaced at 6.0 in. min. on Center with 2.0 in. min. Clear Cover and f'c = 4.5 ksi

#4	#5	#6	#7	#8	#9	#10	#11
1'-10"	2'-3"	3'-4"	3'-11"	4'-5"	5'-6"	6'-10"	8'-2"

Table 5-4: Minimum Lap Length for Epoxy-Coated Slab, Wall, or Footing Bars Spaced at 6.0 in. min. on Center with 1.0 in. min. Clear Cover and f'c = 4.5 ksi

#4	#5	#6	#7	#8	#9	#10	#11
2'-3"	3'-4"	4'-7"	5'-11"	7'-5"	9'-0"	10'-11"	12'-11"

For the same size bar in both top and bottom mat, the more conservative of the two tables shall be shown for ease of construction inspection. Table 5-4 lap splice values may be shown on the deck reinforcing sheet as applicable for both top and bottom mats of reinforcing bars, conservatively. The Designer may also choose to individually detail lap splices for deck rebar to take advantage of the smaller lap lengths required for top slab bars.

All other required lap lengths shall be detailed in the contract plans. Appendix 5A contains design aid tables for calculating development and lap splice lengths for reinforcing not meeting the criteria of Table 5-3 or Table 5-4.

5.4.3 Clear Cover

Concrete cover to main reinforcing bars shall be provided per AASHTO Table 5.10.1-1 and its accompanying notes, except as modified herein. For ***AASHTO 5.10.1*** minimum clear cover specified in the table, use “severe to moderate exposure” for all cases excepted as noted in this section.

- The AASHTO provision for reducing concrete cover in the table by 0.5 in. for stirrups and ties shall apply only to precast girder faces and the minimum clear cover for precast girder faces shall be 1.5 in. or as shown in the worksheets.
- The minimum cover for reinforcing steel for concrete cast against earth shall be 3 in. for uncoated, epoxy coated, or galvanized bars.
- For CIP slabs not cast against earth or CIP deck bottoms, 1 in. minimum cover shall be used.
- For CIP piles, use “corrosive environments” for all cases.
- For drilled shafts on bridges, refer to Table 5-5 for the minimum required cover. The increased covers are adopted from FHWA’s recommendations due to constructability issues that may occur when lesser values of cover are specified for large diameter caissons.
- For elements with rustications, such as columns or abutments, required cover at innermost face of rustications may be reduced by 0.5 in.

Table 5-5: Minimum Clear Cover for Drilled Shafts on Bridges

Drilled Shaft Diameter, D (ft.)	Minimum Concrete Cover (in.)
$D \leq 3$	3
$3 < D < 5$	4
$D \geq 5$	6

5.4.4 Spacing

Reinforcement spacing requirements shall be per AASHTO, except as modified herein. ***AASHTO 5.10.3***

Mild reinforcing bars shall have minimum clear spacing of at least 2 in. for both CIP and precast members (this includes bundled and lapped bars) unless noted otherwise in worksheets. This deviation from AASHTO results from past concrete consolidation issues encountered in Colorado.

5.4.5 Corrosion Protection Requirements

Reinforcing in structural elements that may be subjected to anti-icing or deicing chemicals shall be corrosion resistant (epoxy-coated mild steel, stainless steel, or CRAS). This includes, but is not limited to, all layers of reinforcing in the following elements and bars projecting therein:

- All deck slabs, approach slabs, CIP slab superstructures, and top flanges of CIP box girder bridges used as decks, regardless of wearing surface provided
- Concrete box culvert (CBC) top slabs with 2 ft. or less fill on top

- All abutment and pier diaphragms, abutment caps, and abutment wingwalls
- Pier caps and columns located under an expansion joint
- Retaining wall elements and pier columns located within the splash zone
- Ends of girders within 8 ft. of an expansion joint

5.4.6 Splash Zone Definition

The splash zone extends 10 ft. from the edge of the roadway shoulder, as shown in Figure 5-1 and includes the deck and superstructure elements from above.

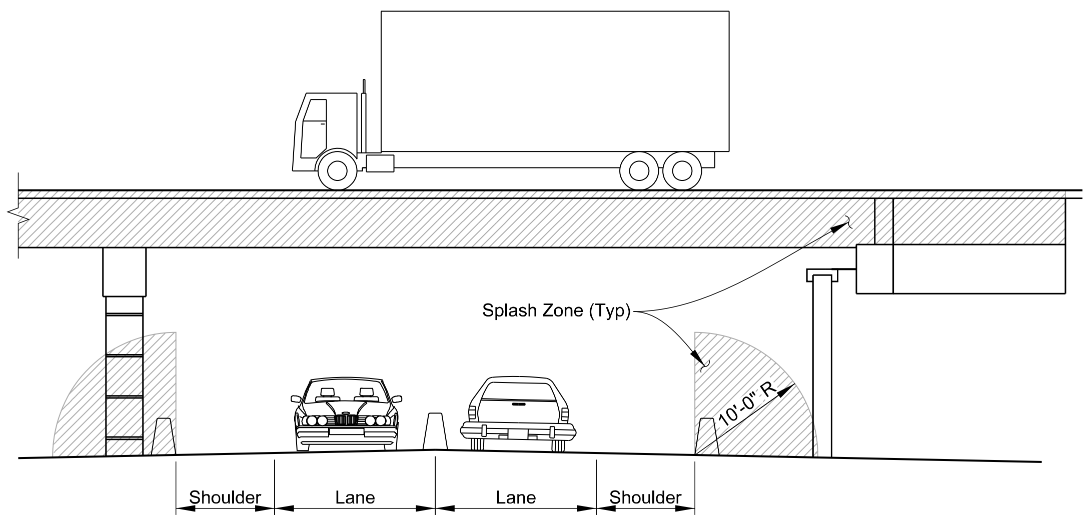


Figure 5-1: Splash Zone

5.4.7 Crack Control Factors

When calculating maximum spacing for crack control, an exposure factor of 0.75 shall be used for reinforcement that is required to be corrosion resistant, except for decks. For all other reinforcement, including decks complying with the wearing surface requirements of Section 9 of this BDM, 1.0 may be used. ***AASHTO 5.6.7***

5.4.8 Mass Concrete

Large volumes of concrete sometimes have an increased potential to generate heat resulting in temperature-related cracking. This is typically an issue for concrete placements with least dimension greater than 6 ft., including, but not limited to, spread footings, thick walls, or bridge piers. In such cases, the Designer should consider requiring the Contractor to submit a thermal control plan. See ACI Manual of Concrete Practice Publication 207 for more information.

5.4.9 Seismic Detailing

Per AASHTO, for bridges in Seismic Zone 1 where S_{D1} is greater than or equal to 0.1, seismic detailing of columns and caissons shall be required for transverse reinforcement in potential hinge zones. When seismic detailing is ***AASHTO 5.11.2*** required for round columns or caissons, spirals are preferred over seismic hoops.

5.4.10 Drilled Shaft and Round Column Shear Reinforcing

For shear reinforcing within drilled shafts and round columns that does not require seismic detailing per BDM Section 5.4.9, hoops containing a lap splice are generally more economical than spirals in the CDOT market.

5.4.11 Pier Cap Reinforcing Details

Cap reinforcement shall be placed below both mats of slab steel and below the main girder reinforcement in mildly reinforced girder bridges. In post-tensioned bridges, the cap reinforcement shall be placed below both mats of slab steel or between the mats of slab steel, if necessary, to provide clearance for post-tensioning ducts.

Hooks on integral cap shear stirrups shall be bent away from the centerline of the cap. The hooks shall enclose a cap reinforcement bar and the stirrups shall be adequately developed. To ensure proper concrete cover for stirrup hooks, hooks shall be below the top mat of slab steel. Figure 5-2 and Figure 5-3 provide details.

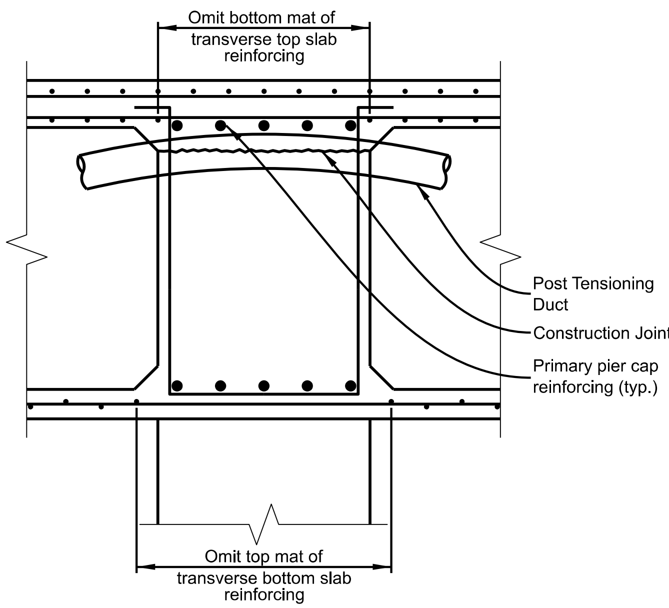


Figure 5-2: Pier Caps in Post-Tensioned Bridges with a Skew Angle of 20 Degrees or Less and Deck Reinforcing Parallel to Cap

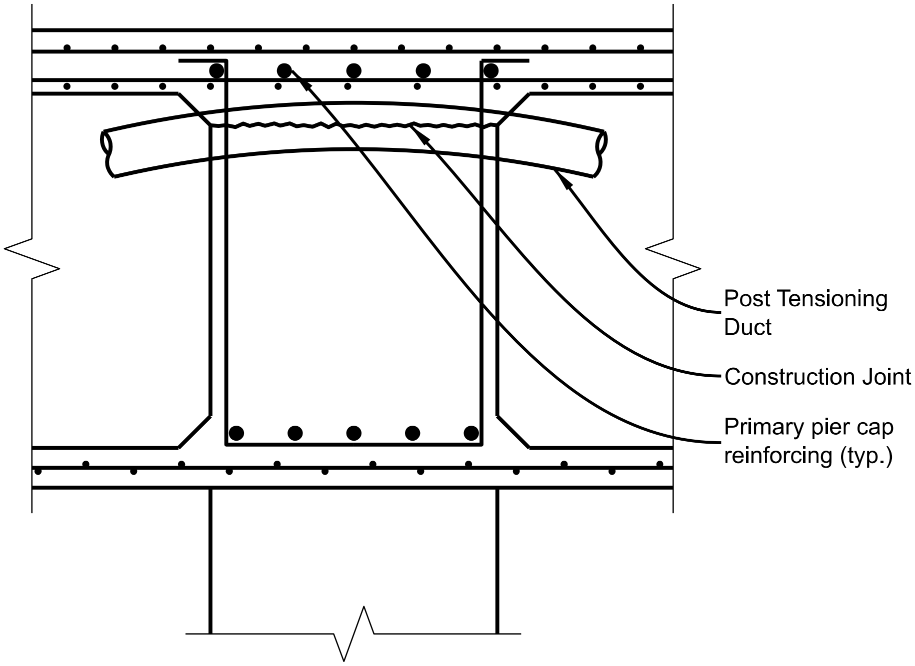


Figure 5-3: Pier Caps in Post-Tensioned Bridges with a Skew Angle Greater Than 20 Degrees and Deck Reinforcing Not Parallel to Cap

For precast girder bridges, cap reinforcement shall be enclosed in closed stirrups, as shown in Figure 5-4 and Figure 5-5. Stirrups shall be adequately developed.

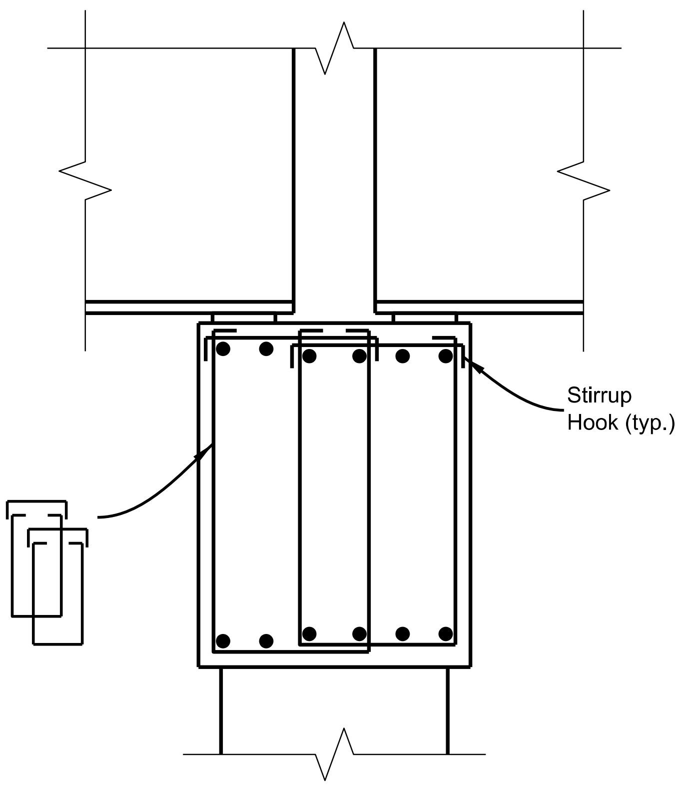


Figure 5-4: Pier Caps in Precast Girder Bridges with Constant-Depth Cap

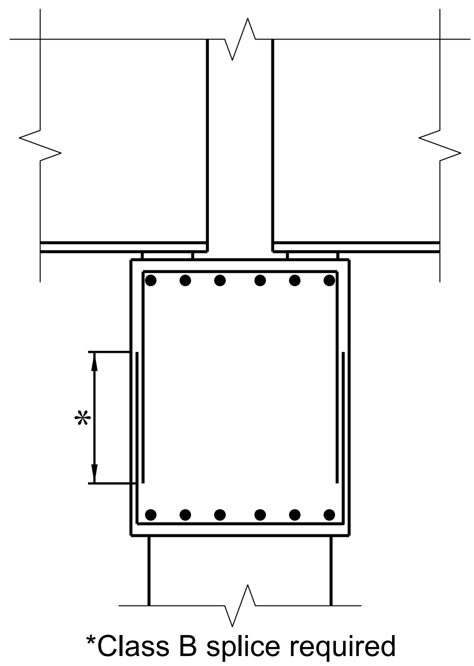


Figure 5-5: Pier Caps in Precast Girder Bridges with Variable-Depth Cap (side steel not shown for clarity)

5.4.12 Combination of Flexural and Axial Effects

Members subjected to flexure and compression may be analyzed using the method of creating an influence diagram using equilibrium and strain compatibility. Many commercial structural design software programs use this approach to create interaction diagrams. Alternatively, AASHTO approximate expressions may be used. ***AASHTO 5.6.4.5***

5.5 PRESTRESSING

5.5.1 General

5.5.1.1 Transformed Section Properties and Elastic Gains ***AASHTO 5.9.1.3 and C5.9.3.2.3a***

AASHTO allows the use of transformed section properties. The Designer should note that when calculating concrete stresses using transformed section properties, the effects of losses and gains due to elastic deformations are implicitly accounted for. Commercial software that calculates elastic gains separately in conjunction with using transformed section properties shall not be used.

Prestressed concrete components designed using the refined estimates of time-dependent losses as specified in AASHTO in conjunction with taking advantage of the elastic gain shall use the increased SVC III live-load factor of 1.0. This increased live load factor also applies to designs using transformed section properties since elastic gains from live load are implicitly accounted for. When elastic gains are not taken advantage of, a live-load factor of 0.8 may be used for SVC III. ***AASHTO 3.4.1***

If elastic gains due to slab shrinkage are taken advantage of, the corresponding girder moment due to slab shrinkage shall be considered in the girder stress calculations. Alternatively, the slab shrinkage elastic gain and the corresponding girder moment may be disregarded. ***AASHTO 5.9.3.4.3d***

5.5.1.2 Intermediate Diaphragms

The Construction Layout sheet shall show the location of intermediate diaphragms for CBT girders.

The Designer is responsible for providing a design that considers stability at the AASHTO Strength III limit state of the girders during construction, especially the stability of exterior girders that may be exposed to wind loads before the deck pour. Additional diaphragms or modifications to CDOT's standard diaphragm details may be needed for special situations. Additional diaphragms or modifications to the standard details should not be used unless determined necessary by calculation.

The Designer should check that the resultant of factored construction loads falls within the area of the leveling pad and that the compression in the portion of the pad loaded in these cases is less than the pad strength. If the resultant falls outside the pad or if the compression strength of the pad is exceeded, additional diaphragms should be provided to reduce eccentricity by causing the girders to overturn in concert.

5.5.1.3 Concrete Stresses

Girders shall be designed such that there is no tension in the concrete under dead load acting alone, at service limit state, and after losses. This provision applies to the pre-compressed tensile zones only as required by the AASHTO tensile stress limits in prestressed concrete. The top ends of girder for a simple span, or simple made continuous bridge, are often under long term tension caused by the prestressing. However, the section is not a pre-compressed tensile zone, so the no tension limit does not apply.

Per AASHTO, compression stresses shall be limited to 0.65 f'c at release. This provision is cited in the BDM due to it being a relatively recent change in AASHTO. ***AASHTO 5.9.2.3.1a***

5.5.1.4 Design Jacking Force

The maximum design jacking force in all prestressing strands (pretensioned or post-tensioned) shall be no more than 75 percent of the ultimate tensile strength of the strand.

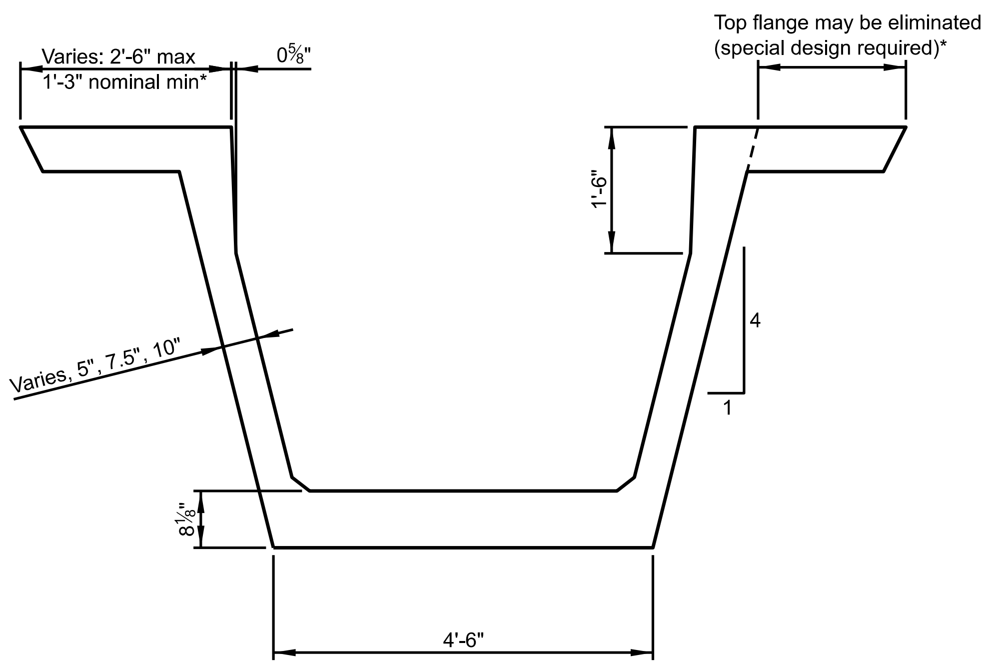
5.5.1.5 Standard Girder Shapes

Table 5-6 identifies the standard properties of CBT girders. See CDOT standard girder worksheets for standard CBT girder dimensions.

Table 5-6: Standard CBT Properties

Section	Depth	A (sq. in.)	Ix (in.^4)	Yb (in.)
CBT37.5	37.5	792	151579	18.5
CBT45	45	845	240424	22.08
CBT54	54	908	378473	26.40
CBT63	63	971	553233	30.74
CBT72	72	1034	767268	35.10
CBT81	81	1097	1023130	39.48
CBT90	90	1160	1323390	43.87

Figure 5-6 identifies the standard dimensions of precast U girders.



*When setting the top flange width of U girders, the Designer shall consider the loss of concrete width for interface shear resistance due to the support requirements for partial depth precast deck panels. While the top flange of U girders may be eliminated entirely from a fabrication standpoint, the limited remaining interface width may preclude using partial depth precast deck panels.

Figure 5-6: Standard U Girder Dimensions

Leap bridge concrete software girder library files are located in CDOT Bridge homepage under Bridge Manuals & Documents section. Designers should contact local suppliers for the following information, which may vary by supplier:

- Pretensioned strand locations
- U girder radius limitations

- U girder height options
- Non-standard CBT girder height options
- U girder and CBT girder thickened bottom flange options
- U girder anchorage blister options

The maximum harped strand height for CBT girders is generally 60” to 66” and is dependent on the girder length/precast bed configuration. For fabrication efficiency, debonded strands are preferred over harped strands and girders less than 100 feet should not use harped strands.

For skewed bridges, the ends of CBT should not be skewed, but the top flange may be clipped to maintain clearance. Wider caps and diaphragms may be considered.

End blocks shall be used for box girders. End blocks are not required for typical applications of the CBT or U girders, but an internal diaphragm of some type is required at the ends of U girders to deal with bearing loads and splaying loads from self-weight and handling.

For box girders harped designs shall not be used. Provide designs with debonded strands only. For skewed bridges, skew the ends of the box girders.The transverse reinforcing steel area in precast box girder flanges shall, as a minimum, be equal to the minimum required shear reinforcing steel for one web. If the top flange of the box is intended to serve as precast stay-in-place formwork for the final deck, this reinforcing shall be designed as the bottom mat of the deck.

5.5.1.6 Maximum Stirrup Spacing

Maximum stirrup spacing in prestressed girders shall be 18 in. ***AASHTO 5.7.2.6***

5.5.1.7 Negative Moment Reinforcement

AASHTO C5.6.3.2.1

For simple made continuous bridges and spliced bridges, the negative moment reinforcing shall be sized for the moment at face of support. The face of support varies depending on pier details and shall be assumed as follows:

- For integral pier caps, the face of support is the face of pier cap.
- Where pier diaphragms are integrally connected to the pier cap, the face of support is the face of diaphragm.
- For pier diaphragms that use the typical CDOT pin detail with a single line of dowels between it and the pier cap, the face of support shall be taken as the centerline of pier.
- For other situations, the Designer is responsible for determining the appropriate face of support.

Longitudinal reinforcing for negative moment placed near the top of deck may be accomplished one of two ways:

- Continuing the typical top longitudinal deck steel over the pier and bundling to the typical bars with larger bars where needed.
- Discontinuing the top longitudinal deck steel and continuing with larger bars where needed. Two bar bundles may be used for the peak negative moment region for this option.

When partial depth precast deck panels are permitted on the project, bottom longitudinal reinforcing in the deck shall not be used for composite girder negative moment capacity calculations.

See Section 9 of this BDM for the minimum clearance required between deck reinforcing and the top of partial depth precast deck panels.

5.5.1.8 Shipping and Handling

Per AASHTO, the fabricator is responsible for the shipping and handling design. However, when the Designer specifies temporary girder support locations on the plans, the Designer is responsible for designing the girder for the force effects resulting from that support condition. ***AASHTO 5.12.3.2.1***

5.5.1.9 Shipping Weights and Lengths

For typical locations along the Front Range urban corridor, typical maximum girder length and shipping weights are 154 ft. and 240 kip, respectively. For lengths or weights exceeding these limits and for project site locations where delivery routes may have constraints, such as sharp curved roads and/or tunnels, the Designer shall coordinate with local suppliers to determine the dimensional and weight limitations of the proposed girders.

5.5.1.10 Partial Prestressing

Partial prestressing is not addressed in AASHTO. Partial prestressing as a design strategy may be allowed with approval from Unit Leader and concrete SMEs.

Partial prestressing refers to situations where the prestressing is insufficient to reduce flexural tensile stresses to the Service III or temporary tensile stress limits. When partial prestressing is used, expected crack openings shall be controlled to an appropriate limit in the Service I load case. This control may be provided by distribution of bonded reinforcement with an area of at least 1 percent of the area of the tensile zone or by limiting tensile stresses or tensile strains. Also, when partial prestressing is used, live and dead load deflections shall be calculated using the appropriate cracked section properties. Strength shall be checked in all relevant load cases, including construction and handling loads. In the instance of partial prestressing, either compressive stress limits may be applied at the service loads or ultimate strength limits may be applied.

For sheltered locations not subject to deicing salts, rain, snow, or direct sunlight, 0.024 in. may be an acceptable crack opening at the reinforcing depth. For locations subjected to the above elements, 0.016 in. may be taken as an acceptable crack opening.

5.5.2 Pretensioned Concrete

5.5.2.1 Girder Haunch, Camber, and Dead Load Deflections

A. General

The Designer is responsible for setting the thickness of the haunch at supports, such that an adequate haunch is maintained along the length of the girder considering the estimated girder camber with tolerance, dead load deflections, deck profile grade and cross slope, and required precast deck panel clearance when applicable.

For side-by-side box or slab girders, the haunch is synonymous with the deck. In this case, the Designer is responsible for setting the deck thickness at supports and verifying that adequate deck thickness is maintained along the length of girder, considering the applicable factors noted previously for girder haunches.

B. Minimum Haunch

The minimum haunch at supports shall be 1.5 in. where partial depth precast deck panels are permitted. This allows the required 1 in. vertical clearance underneath the panels, plus 0.5 in. of tolerance that accounts for girder depth variation and/or bearing seat height variability. Where partial depth precast deck panels are not permitted, the minimum haunch at supports shall be 0.5 in.

The minimum estimated haunch between supports shall be 1 in. where partial depth precast deck panels are permitted and may be taken as zero where partial depth precast deck panels are not permitted.

For side-by-side box or slab girders, the minimum deck thickness specified at supports shall be 5 in., in accordance with Section 9.5 of this BDM. The minimum estimated deck thickness between supports shall also be maintained at 5 in.

All minimum haunch requirements above must be met along the entire width of the top flange of the girder, not only at centerline. The Designer must take into consideration cross slope effect on the haunch variance.

C. Maximum Haunch

There is no limiting maximum haunch either at supports or for the estimated haunches between supports. For haunches with a side face dimension estimated at 8 in. or greater, minimum temperature and shrinkage reinforcement shall be added to the side faces of the haunch. ***AASHTO 5.10.6***

D. Camber Estimates

Release and erection cambers should be estimated using the plan specified concrete design strength minimums per Section 5.3.1.2.

When using camber calculations where the age is a factor for the camber at the time of deck pour, the age of the girder shall be assumed to be 60 days. See Section 5.7.2 for situations where this age may be assumed to be less than 60 days.

Tolerances for girder cambers with respect to estimating minimum and maximum haunches are as follows:

- For CBT girders, camber shall be assumed to be up to 20 percent over or 50 percent under the predicted camber.
- For slab and box girders, camber shall be assumed to be up to 50 percent over or 50 percent under the predicted camber.
- For design cambers greater than 1”, the camber tolerance shall be taken as no less than ± 1 in. For camber designs less than 1”, camber tolerance may be less than 1” with approval by Unit Leader. This is typically restricted to widening with site constraints.

These camber tolerances are based on fabrication data for girders with cambers greater than 1” and a 90% confidence range on the envelopes as shown in Table 5-7. Data for design cambers under 1” have a higher camber variance.

If a girder line can be eliminated by reducing over and/or under camber tolerances, a variance shall be requested.

Table 5-7: Camber Data

Girder Type		CBT Girder	BX Girder
Count		192	871
Minimum		-44.44%	-90.33%
Maximum		55.96%	209.09%
Mean		5.85%	-3.75%
Standard Deviation		19.18%	32.10%
Confidence Range	99.7%	63.4% / -51.7%	92.6% / -100.1%
	99%	55.2% / -43.5%	78.9% / -86.4%
	95%	43.4% / -31.7%	59.2% / -66.7%
	90%	**37.4% / -25.7%**	**49.1% / -56.6%**
	85%	33.5% / -21.8%	42.5% / -50.0%

E. Dead Load Deflections

Dead load deflections may be calculated assuming no long-term increase in deflection beyond construction. This assumption may be used for the dead load deflection reported on the girder sheet, for estimating haunches, and for setting deck grades. The long term multipliers shall not be used in the girder design.

F. Deck Profile Grade Effect

The deck profile grade ordinate shall be taken as the difference between a chord of profile grade from bearing to bearing and the actual profile grade at any point along the chord line.

This ordinate will add to the haunch thickness if profile grade is higher than the chord line and, conversely, will subtract from the haunch thickness if profile grade is lower than the chord line.

G. Design Considerations

Side-by-side box girders shall be designed for service and strength criteria using the range of deck thicknesses expected considering the assumed tolerances for box girder cambers. The dead load deflection reported on the plans and used to set deck grades shall be calculated with the deck thickness resulting from the predicted girder camber. The deck concrete quantity may also be based on this deck thickness. The use of our 5" minimum deck over the girder allows the designer to utilize the distribution factors based on F type (AASHTO Table 4.6.2.2.1-1) girder arrangements per AASHTO 5.12.2.3.3(f) so shear keys are generally not allowed.

Other girders may be conservatively designed assuming the maximum estimated haunch due to an under-cambered girder for all calculations.

Girder sag is not permitted for any girder type, unless there is prior approval by the Unit Leader. Sag is considered prevented when the girder camber remaining after deducting the under-camber tolerance and the dead load deflection is greater than or equal to zero. The dead load deflection used for this check need not be magnified by long-term effects. For side-by-side boxes, the dead load deflection for this check shall be based on the increased deck weight resulting from the girder being under-cambered.

In lieu of considering over-camber tolerance in the design of side-by-side box girder bridges, the bearing seats may be lowered by the over-camber tolerance amount. Shims shall be provided where the total shim stack height equals the over-camber tolerance amount. If the girders are over-cambered, shims may be removed as necessary to maintain a 5 in. minimum deck thickness.

A weighted average haunch (or slab depth for side-by-side boxes) may be used for dead load calculations for girder design. The equation below is derived for the midspan moment effect assuming the haunch (or slab) varies parabolically with the apex (either concave or convex) at midspan:

$$(D_1 + 10*D_2 + D_3) / 12 \qquad \text{Eq. 5.1}$$

A volume-based average haunch (or slab depth for side-by-side boxes) may be used for the concrete quantity. The equation below is derived assuming the top of girder is chorded between the end of girder and midspan:

$$(D_1 + 2*D_2 + D_3) / 4 \qquad \text{Eq. 5.2}$$

Where D_1 is the depth over one bearing, D_2 at midspan, and D_3 over the other bearing.

See Example 7 for detailed examples of setting girder haunches and verifying the above criteria.

5.5.2.2 Hold Down Limits

Harped strands shall be designed so that the hold-down force does not exceed 4.0 kip per strand.

5.5.2.3 Partially Debonded Strands

If debonding is used, the design shall follow the debonding criteria outlined in AASHTO 5.9.4.3.3.Minimum Plan Requirements ***AASHTO 5.9.4.3.3***

The contract plans for pretensioned members shall specify:

- Jacking force
- Area of prestressing steel
- Minimum concrete strength at jacking and at 28 days
- Center of gravity of prestressing force path
- Final force
- Dead load deflection
- Expected cambers (release and before deck pour)
- Estimated haunch at midspan (estimated deck thickness for side-by-side box girders)

5.5.3 Post-Tensioned Concrete

5.5.3.1 Anchorages

The post-tensioning supplier is responsible for the design of the local zone, including the anchorage device itself and confinement reinforcement. The Design Engineer is responsible for all other anchorage-related designs, including the general zone. The Designer shall verify that all anchorage design assumptions are correctly represented on the plans to aid the supplier in the design of the local zone to coordinate with the design of the general zone. ***AASHTO 5.9.5.6***

Composite anchorages shall not be permitted. Multi-plane anchorages may be used. ***AASHTO 5.4.5***

The design jacking force of strands shall be 75 percent of the ultimate tensile strength of the tendon for the design of the post-tensioned member. For the design of anchorages, including the local and general zones, the anchorage force shall be based on 80 percent of the ultimate tensile strength of the tendon. This allows reserve capacity for increasing the jacking force to the AASHTO limit, if needed, during construction. ***AASHTO 5.9.2.2***

The plans shall show the configuration of the anchorages and the arrangement of ducts at typical high and low points appropriate for the duct and strand size noted on the plans. The arrangement of anchorages shall permit a center-to-center anchorage spacing of at least $\sqrt{(2.2P_j / f'_{ci})}$ in. and a spacing from the center of each anchorage to the nearest concrete edge of at least half that value. If web flares are needed for this arrangement, they shall be dimensioned in the plans and included in the quantities.

5.5.3.2 Post-tensioning Ducts

A. Spacing

- Minimum clear spacing of ducts shall be the greater of 40 percent of the nominal duct diameter or 1.5 in. ***AASHTO 5.9.5.1***
- Bundled ducts shall not be used without approval from Unit Leader in coordination with the Fabrication/Construction Unit.

AASHTO 5.10.1

B. Clear Cover

- For cast-in-place bridges, the minimum clear cover to ducts shall be the greater of 75 percent of the nominal duct diameter or 3 in.
- For precast girder bridges, the minimum clear cover to ducts shall be the greater of 50 percent of the nominal duct diameter or 2 in. An exception to this is post-tensioned CBT girders, which have demonstrated good past performance with a minimum of 1.75 in. clear cover.
- Clear cover for ducts curved in plan shall meet the greater of the applicable criteria above, or the confinement criteria as specified in AASHTO. ***AASHTO 5.9.5.4.3***

C. Eccentricity ***AASHTO 5.9.1.6***

Eccentricity of strand within ducts shall be considered when modeling the tendons. In lieu of using the eccentricities specified in AASHTO Figure C5.9.1.6-1, manufacturer-specific eccentricity may be used if known during design.

5.5.3.3 Monostrands

Monostrand tendons shall be of a fully encapsulated waterproof construction whether permanent or temporary.

Permanent monostrand tendons placed in any of the locations listed below shall be of a type certified by their manufacturer for chloride contaminated environments:

- In decks or haunches above girders
- When any part of the tendon is within a horizontal distance equal to the structure depth of an expansion joint or within 6 in. of the back face of an integral abutment
- When tendon is used in below ground construction

Monostrands and bundles of up to 4 monostrands in plant produced members using a highly fluid small aggregate concrete, or using a moderately fluid small aggregate concrete with form vibrators, shall have a clear spacing of at least 1.25 in.

Field produced members or members not using form vibrators or a fluid small aggregate concrete shall have a clear spacing between monostrands or bundles of monostrands of at least 1.5 in.

5.5.3.4 Unbonded Tendon Redundancy

An unbonded tendon is any tendon that is not bonded to the structure throughout its length in its final installed condition. Common examples are monostrand tendons and multi-strand tendons used in externally post-tensioned precast segmental box girders.

For each girder, any two unbonded tendons shall be assumed to be failed. The moment strength provided by the remaining tendons and reinforcement shall be at least 80 percent of that required by the Strength I load combination. The same provision applies to any 13.5 ft. width of slab. The 13.5 ft. limit is a conservative limit based on the arching capability of the slab.

At the discretion of the Designer, for longitudinal tendons in multiple girder systems in which there are adequate load paths between the girders, the entire connected cross section may be considered a single girder element.

5.5.3.5 Transverse Post-Tensioning in Adjacent Precast Box Girders

For railroad bridges, adjacent box girders without a CIP deck are permitted with the use of transverse post-tensioning, per AREMA. For the design of adjacent box girders without a CIP deck, the design guidance of PCI Bridge Design Manual Section 8.9 may be used.

Adjacent box girders without a CIP deck are not permitted for traffic bridges. For normal traffic bridges utilizing adjacent box girders, shear keys shall not be used. The use of our 5" minimum deck over the girder allows the designer to utilize the distribution factors based on F type (AASHTO Table 4.6.2.2.1-1) girder arrangements per AASHTO 5.12.2.3.3(f).

5.5.3.6 Severe Exposure Category for Tension Limits

The following locations shall be considered the severe exposure category for AASHTO concrete tension limits: ***AASHTO 5.9.2.3.2b***

- Tops of decks that are post-tensioned
- Top flanges acting as the deck for CIP post-tensioned girders

Tops of girders for post-tensioned spliced bridges need not be classified as severe exposure.

5.5.3.7 Through-the-Thickness Web Reinforcing

Through-the-thickness reinforcing equal to 9 percent of the area of the tendon shall be provided for the following situations:

- To arrest propagation of through-the-thickness cracks driven by misalignment at construction joints. The specified amount of reinforcing shall be placed near each tendon passing through a joint. The reinforcement shall be split between each side of the joint.
- Due to potential through-the-thickness forces at thickness transitions at the beginning and end of transitions of web or flange thickness. The specified amount of reinforcing shall be located near the tendon and split between each side of the transition beginning or end.

Through-the-thickness reinforcement shall be anchored as close to the face of the concrete as practical. Headed studs or stud-rails may be used in lieu of reinforcing.

Figure 5-7 and Figure 5-8 illustrate through-the-thickness reinforcement.

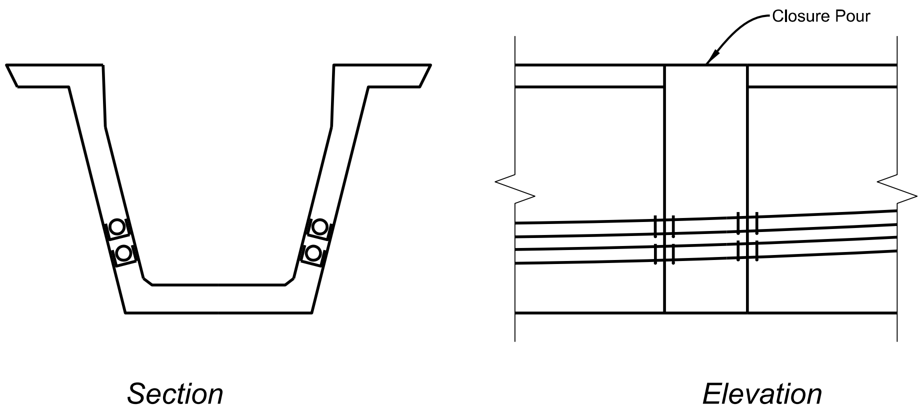


Figure 5-7: Through-the-Thickness Steel at Construction Joints

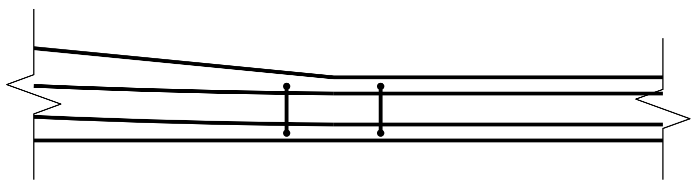


Figure 5-8: Plan View of Through-the-Thickness Reinforcing at a Web Thickness Transition

5.5.3.8 Horizontally Curved Tendons

Reinforcing requirements for horizontally curved tendons shall be per AASHTO. ***AASHTO 5.9.5.4.3***

5.5.3.9 Minimum Plan Requirements

The contract plans for post-tensioned members shall specify:

- Jacking force
- Area of prestressing steel
- Minimum concrete strength at jacking and at 28 days
- Center of gravity of prestressing force path
- Jacking ends
- Anchor sets
- Friction constants
- Long-term losses assumed in the design
- Strand and duct size assumed in the design
- Net long-term deflections and expected cambers

- Estimated haunches at midspans (for spliced girders only)

5.6 LONGITUDINAL REINFORCEMENT FOR SHEAR

5.6.1 General

AASHTO 5.7.3.5

AASHTO Equation 5.7.3.5-1 accounts for increased tension in longitudinal reinforcement caused by shear. The applicability of this interaction equation depends on support and loading conditions. This section is provided as further clarification of AASHTO.

5.6.2 Direct Loading and Supports

Direct loading may be assumed where the load introduces compression directly onto the compression face of the member.

Direct supports may be assumed where the reaction introduces direct compression directly onto the compression face of the member.

In simple-made-continuous bridges, pier caps that are detailed as pinned connections to the pier diaphragms may be classified as direct supports.

Figure 5-9 presents examples of direct support and direct loading conditions.

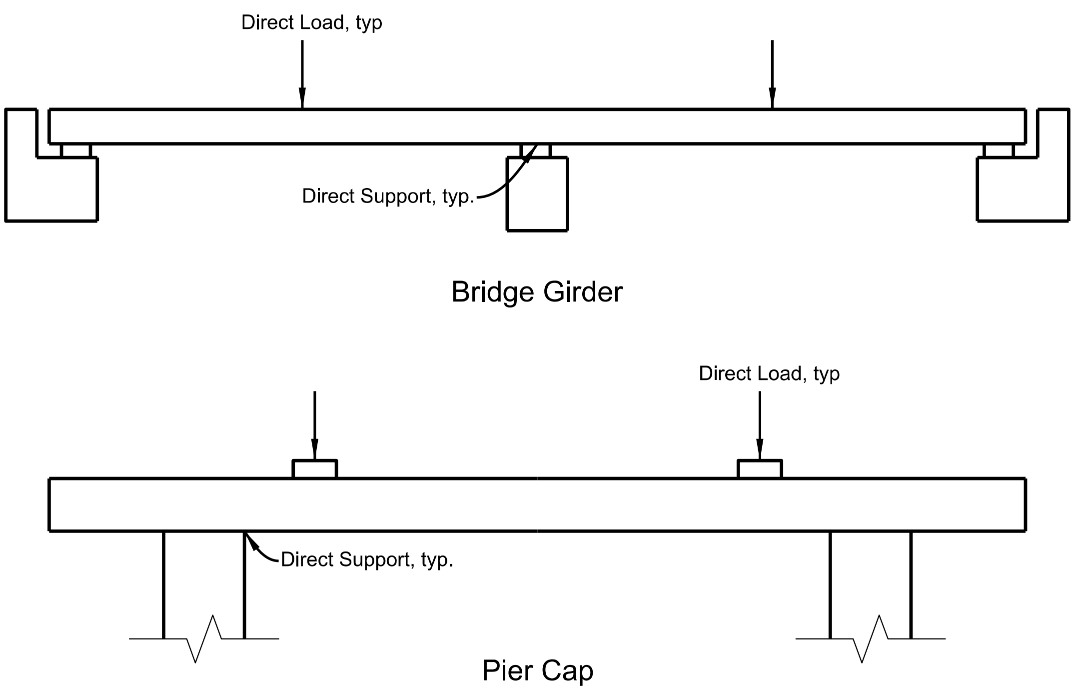


Figure 5-9: Examples of Direct Supporting and Loading Conditions

For direct support/loading conditions, the following provisions apply:

- Checking interaction is not required at or near direct supports or at other locations of maximum moment, such as at or near midspan. At these locations, the longitudinal reinforcement needed for moment demand alone need not be exceeded.

- Interaction shall be checked where longitudinal reinforcement is reduced along the member. In this case, the equation shall be checked at 10th points that are away from the maximum moment locations and at locations of reinforcement reduction.
- In summary of the previous two notes, if the maximum needed flexural reinforcement is continuous through the member and not reduced, checking the interaction equation is not required.

5.6.3 Indirect Loading and Supports

Any load or support that is not classified as a direct load or support shall be classified as indirect. For practical purposes in bridge design, this primarily happens at integral pier caps.

In simple-made-continuous bridges, pier caps that are detailed as integrally connected to the pier diaphragm shall be assumed to be indirect supports.

Figure 5-10 shows an example of an indirect support. In this case, the girders shall be considered indirectly supported, and the pier cap shall be considered indirectly loaded. The pier cap may be considered directly supported by the columns.

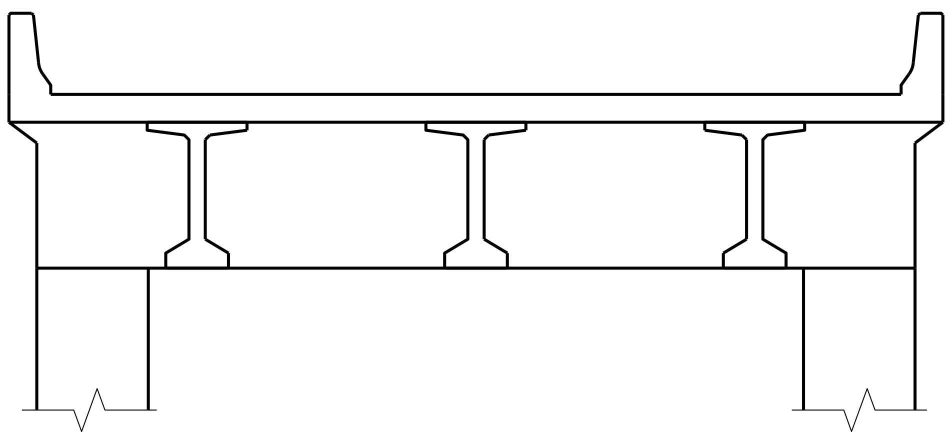


Figure 5-10: Indirect Support/Loading – Integral Pier Cap

For indirect support and loading conditions for a typical integral pier cap, the following provisions apply:

- Interaction shall be checked in the girder at the face of the integral pier cap, at 10th points, and at places of rebar termination.
- Interaction does not need to be checked in the girder at midspan if it is directly loaded.
- Interaction shall be checked in the pier cap at points of maximum positive moment, at 10th points, and at locations where positive moment reinforcement is terminated.
- Interaction need not be checked in the pier cap at or near the face of column, as this is at a direct support. But if negative moment reinforcement

is reduced, then interaction shall be checked at 10th points away from the direct support and at locations of rebar termination.

5.6.4 Simply Supported Girder Ends

AASHTO Equation 5.7.3.5-2 shall be satisfied at the inside edge of the bearing area of simple supports. Girders supported by the typical CDOT integral abutment are required to meet this provision.

5.7 SIMPLE SPAN PRESTRESSED MADE CONTINUOUS

5.7.1 General

These provisions apply to multi-span bridges composed of simple-span precast girders with continuity diaphragms cast with the deck between ends of girders at interior supports. These bridges shall be designed using the specific provisions for this structure type per AASHTO, except as amended herein. ***AASHTO 5.12.3.3***

5.7.2 Age of Girder When Continuity Is Established

The plans shall specify the minimum age of the precast girder when continuity is established (deck and continuity diaphragm placed). ***AASHTO 5.12.3.3.4***

For standard designs, the minimum age before establishing continuity shall be 60 days. If waiting 60 days for deck/diaphragm placement has negative impacts to the project schedule, the minimum age may be specified as less than 60 days. In either case, the following simplifications shall apply:

- Positive restraint moment caused by girder creep and shrinkage and deck slab shrinkage shall be taken to be zero.
- Computation of restraint moments shall not be required.
- A positive moment connection shall be designed to resist 1.2*M_{cr}.

5.7.3 Degree of Continuity at Various Limit States

The connection between precast girders at a continuity diaphragm may be considered fully effective if the plans require that the age of the precast girders be at least 60 days before deck/diaphragm pour. ***AASHTO 5.12.3.3.5***

If the precast girder connection between precast girders at a continuity diaphragm does not satisfy this requirement, the joint shall be considered non-effective.

Superstructures with fully effective connections at interior supports may be designed as fully continuous structures for all loads applied after continuity is established for both service and strength limit states.

Superstructures with non-effective connections at interior supports shall have designs enveloped for the worst-case force effects between simple span and continuous behavior for all loads applied after continuity is established for all limit states. For example, simple span behavior will govern positive moment regions, and continuous behavior will govern negative moment regions.

The provisions in AASHTO for partially effective continuity diaphragms shall be disregarded.

5.8 PRECAST SPLICED BRIDGES

5.8.1 General

Precast spliced bridges are structures using precast girders fabricated in segments that are joined or spliced longitudinally to form girders in the final structure. These bridges shall be designed using the specific provisions for this structure type per AASHTO, except as amended herein. *AASHTO 5.12.3.4*

5.8.2 Girder Age Restrictions

The contract documents shall show the minimum age of girder segments at the time of post-tensioning. The age may be specified as the average age of segments per girder line.

Where expansion joint movements are at or near the full joint design capacity, the contract documents shall show the minimum time required to elapse after post-tensioning and before installing expansion joints.

5.8.3 Joints Between Segments

AASHTO 5.12.3.4.2

Match-cast joints shall not be used between segments unless approval is obtained from Unit Leader in coordination with the Fabrication/Construction Unit.

5.8.4 Details of Closure Joints

AASHTO 5.12.3.4.2b

The width of closure joints shall not be less than 2 ft.

5.8.5 Segment Design

Where girder segments are handled before the application of prestressing, the provisions of AASHTO 5.6.7 shall apply until post-tensioning is applied. *AASHTO 5.12.3.4.3*

Refer to Section 5.5.1.8 for additional segment shipping and handling design requirements.

5.8.6 Consideration of Future Deck Replacement

To facilitate future deck replacement, the follow criteria shall apply: *AASHTO 5.12.3.4.4*

1. All post-tensioning tendons should be located fully within the girder.
2. All tendons shall be stressed before deck placement.
3. All temporary girder supports shall be removed prior to deck placement.

Deviations from items 2 and 3 may be permitted with approval from Unit Leader in coordination with the State Bridge Engineer. In this case, an analysis of the feasibility of future deck replacement shall be accomplished, and a future deck replacement plan shall be provided in the bridge design plans. The deck replacement plan shall delineate the construction steps necessary for deck replacement including, but not limited to, the following, as applicable:

- Special requirements for deck removal sequencing
- Temporary girder supports required and accompanying reactions

- Additional post-tensioning required (if this is required, accommodations for future post-tensioning shall be detailed into the plans)
- Special requirements for deck placement sequencing

5.8.7 Girder Camber, Haunch, and Dead Load Deflections

The provisions of Section 5.5.2 for pretensioned girder bridges should generally be followed for spliced girder bridges with the following additional considerations.

The total girder camber is the superimposed total of the individual segment camber, the camber resulting from continuity post-tensioning, and the camber induced through the setting of temporary support bottom-of-girder elevations.

The dead load deflection reported on the plans shall include long-term effects. The long-term effects shall be estimated in conjunction with a time-dependent, staged construction analysis method. The long-term dead load deflection shall be used for setting deck grades, setting and estimating girder haunches, and verifying overall girder camber.

CDOT has not experienced the same severity of issues regarding camber variability and associated girder sag for spliced bridges as it has for pretensioned girder bridges. For spliced bridges, the Designer is responsible for determining appropriate camber tolerances used for setting and estimating girder haunches and for verifying adequate final girder camber.

5.9 CAST-IN-PLACE CONCRETE GIRDERS

5.9.1 General

CIP box and T-beam girders constructed on falsework shall be designed using the specific provisions for CIP girders per AASHTO, except as amended herein. ***AASHTO 5.12.3.5***

5.9.2 Box Girder Bottom Slab Slope

Except for crowned roadways, the bottom slab should be made parallel to the top slab. For crowned roadways, the bottom slab should be made horizontal.

5.9.3 Box Girder Formwork Load

Design shall include the additional dead load for deck formwork to be left in place. This formwork load shall be applied over a width equal to exterior web to exterior web.

5.9.4 Web Reinforcement

One-piece “U” stirrups shall not be used in box girder webs.

For post-tensioned girders, each web face shall contain continuous longitudinal reinforcement of at least 0.20 in^2/ft, spaced at 12 in. max.

5.10 SEGMENTAL BOX GIRDERS

5.10.1 General

AASHTO 5.12.5

Segmental box girder bridges are composed of multiple box girder segments where the width of each segment is typically the full width of the bridge. The segments are post-tensioned together longitudinally to act as one continuous structure. Segmental structures shall be designed using the specific structure type provisions per AASHTO, except as amended herein.

5.10.2 Provision for Future Dead Load or Deflection Adjustment

AASHTO 5.12.5.3.9c

The AASHTO provision for detailing segmental structures to accommodate future unbonded tendons that provide at least 10 percent of the positive and negative moment post-tensioning force may be waived in spans for which the long-term deflection is less than 0.5 percent of the span length. Long-term deflection variability can be easily affected by the unpredictability of ultimate creep and shrinkage coefficients, prestressing, losses, and weight; especially if the future wearing surface occurs early. The addition of 10 percent future tensioning to segmental spans with this magnitude of stiffness would not change long-term cambers significantly.

This waiver is contingent upon the bridge being designed for a future wearing surface in accordance with Section 3 of this BDM.

APPENDIX 5A- DEVELOPMENT LENGTH & LAP SPLICE LENGTH DESIGN AIDS

Tables for development length and lap splices are provided for the following cases:

- Table 5A-1: Tension Development Length of Deformed Bars
- Table 5A-2: Tension Development Length of Epoxy-Coated Bars (Coating Factor = 1.5)
- Table 5A-3: Tension Development Length of Epoxy-Coated Bars (Coating Factor = 1.2)
- Table 5A-4: Compression Development Length and Minimum Lap Splices in Compression
- Table 5A-5: Tension Development Length of 90 and 180 Degree Standard Hooks
- Table 5A-6: Class B Tension Lap Splice Lengths of Deformed Bars
- Table 5A-7: Class B Tension Lap Splice Lengths of Epoxy-Coated Bars (Coating Factor = 1.5)
- Table 5A-8: Class B Tension Lap Splice Lengths of Epoxy-Coated Bars (Coating Factor = 1.2)

The Designer is responsible for calculating development lengths and lap splices for situations not covered by these tables.

Table 5A-1: Tension Development Length of Deformed Bars

		Tension Development Length (L_d) of Uncoated Deformed Bars (in.)							
		λ_{rc} = 0.4		λ_{rc} = 0.6		λ_{rc} = 0.8		λ_{rc} = 1.0	
Bar #	L_{db}	Top Bars	Others	Top Bars	Others	Top Bars	Others	Top Bars	Others
3	25.5	14	12	20	16	27	21	34	26
4	33.9	18	14	27	21	36	28	45	34
5	42.4	23	17	34	26	45	34	56	43
6	50.9	27	21	40	31	53	41	67	51
7	59.4	31	24	47	36	62	48	78	60
8	67.9	36	28	53	41	71	55	89	68
9	76.6	40	31	60	46	80	62	100	77
10	86.2	45	35	68	52	90	69	113	87
11	95.7	50	39	75	58	100	77	125	96
14	114.9	60	46	90	69	120	92	150	115
18	153.2	80	62	120	92	160	123	200	154

Notes:

1. Values based on use of normal weight concrete.
2. Top bars are horizontal bars placed so more than 12 in. of fresh concrete is cast below the reinforcement.
3. The minimum tension development length is 12 in.
4. See AASHTO 5.10.8.2.1.

Calculation Variables:

Tension Development Length, L_d = $L_{db}*\lambda_{rl}*\lambda_{cf}*\lambda_{rc}*\lambda_{er}/\lambda$
Basic Tension Development Length, L_{db} = $2.4d_b*fy/sqrt(f'c)$
Reinforcement Location Factor, λ_{rl} = 1.3 For top bars
λ_{rl} = 1.0 For others
Coating Factor, λ_{cf} = 1.0
$\lambda_{rl}*\lambda_{cf}$ = 1.3 For top bars
$\lambda_{rl}*\lambda_{cf}$ = 1.0 For others
Excess Reinforcement Factor, λ_{er} = 1.0
Concrete Density Modification Factor, λ = 1.0
Reinforcing Steel Yield Strength, fy = 60 ksi
Compressive Strength of Concrete, f'c = 4.5 ksi
d_b = bar diameter
Reinforcement Confinement Factor, λ_{rc}: User shall calculate

Table 5A-2: Tension Development Length of Epoxy-Coated Bars (Coating Factor = 1.5)

		Tension Development Length (L_d) of Epoxy-Coated Steel Reinforcing Bars (in.) λ_{cf} = 1.5 (cover less than 3*db or clear spacing between bars less than 6*db)							
		λ_{rc} = 0.4		λ_{rc} = 0.6		λ_{rc} = 0.8		λ_{rc} = 1.0	
Bar #	**L_{db}**	**Top Bars**	**Others**	**Top Bars**	**Others**	**Top Bars**	**Others**	**Top Bars**	**Others**
3	25.5	18	16	26	23	35	31	44	39
4	33.9	24	21	35	31	47	41	58	51
5	42.4	29	26	44	39	58	51	73	64
6	50.9	35	31	52	46	70	62	87	77
7	59.4	41	36	61	54	81	72	101	90
8	67.9	47	41	70	62	93	82	116	102
9	76.6	53	46	79	69	105	92	131	115
10	86.2	59	52	88	78	118	104	147	130
11	95.7	66	58	98	87	131	115	163	144
14	114.9	79	69	118	104	157	138	196	173
18	153.2	105	92	157	138	209	184	261	230

Notes:
1. Values based on use of normal weight concrete.
2. Top bars are horizontal bars placed so more than 12 in. of fresh concrete is cast below the reinforcement.
3. The minimum tension development length is 12 in.
4. See AASHTO 5.10.8.2.1.

Calculation Variables:

Tension Development Length, L_d =	$L_{db}*\lambda_{rl}*\lambda_{cf}*\lambda_{rc}*\lambda_{er}/\lambda$
Basic Tension Development Length, L_{db} =	$2.4d_b$*fy/sqrt(f'c)
Reinforcement Location Factor, λ_{rl} =	1.3 For top bars
λ_{rl} =	1.0 For others
Coating Factor, λ_{cf} =	1.5
$\lambda_{rl}*\lambda_{cf}$ =	1.7 For top bars (max. of 1.7)
$\lambda_{rl}*\lambda_{cf}$ =	1.5 For others (max. of 1.7)
Excess Reinforcement Factor, λ_{er} =	1.0
Concrete Density Modification Factor, λ =	1.0
Reinforcing Steel Yield Strength, fy =	60 ksi
Compressive Strength of Concrete, f'c =	4.5 ksi
d_b =	bar diameter
Reinforcement Confinement Factor, λ_{rc}:	User shall calculate

Table 5A-3: Tension Development Length of Epoxy-Coated Bars (Coating Factor = 1.2)

		Tension Development Length (L_d) of Epoxy-Coated Steel Reinforcing Bars (in.) λ_{cf} = 1.2 (cover at least 3*db and clear spacing between bars at least 6*db)							
		λ_{rc} = 0.4		λ_{rc} = 0.6		λ_{rc} = 0.8		λ_{rc} = 1.0	
Bar #	**L_{db}**	**Top Bars**	**Others**	**Top Bars**	**Others**	**Top Bars**	**Others**	**Top Bars**	**Others**
3	25.5	16	13	24	19	32	25	40	31
4	33.9	22	17	32	25	43	33	53	41
5	42.4	27	21	40	31	53	41	67	51
6	50.9	32	25	48	37	64	49	80	62
7	59.4	38	29	56	43	75	58	93	72
8	67.9	43	33	64	49	85	66	106	82
9	76.6	48	37	72	56	96	74	120	92
10	86.2	54	42	81	63	108	83	135	104
11	95.7	60	46	90	69	120	92	150	115
14	114.9	72	56	108	83	144	111	180	138
18	153.2	96	74	144	111	192	148	240	184

Notes:
1. Values based on use of normal weight concrete.
2. Top bars are horizontal bars placed so more than 12 in. of fresh concrete is cast below the reinforcement.
3. The minimum tension development length is 12 in.
4. See AASHTO 5.10.8.2.1.

Calculation Variables:

Tension Development Length, L_d = L_{db}*λ_{rl}*λ_{cf}*λ_{rc}*λ_{er}/λ
Basic Tension Development Length, L_{db} = 2.4d_b*fy/sqrt(f'c)
Reinforcement Location Factor, λ_{rl} = 1.3 For top bars
λ_{rl} = 1.0 For others
Coating Factor, λ_{cf} = 1.2
λ_{rl}*λ_{cf} = 1.6 For top bars (max. of 1.7)
λ_{rl}*λ_{cf} = 1.2 For others (max. of 1.7)
Excess Reinforcement Factor, λ_{er} = 1.0
Concrete Density Modification Factor, λ = 1.0
Reinforcing Steel Yield Strength, fy = 60 ksi
Compressive Strength of Concrete, f'c = 4.5 ksi
d_b = bar diameter
Reinforcement Confinement Factor, λ_{rc}: User shall calculate

Table 5A-4: Compression Development Length and Minimum Lap Splices in Compression

	Min. Compressive Development Length (L_{db}) (in.)		Min. Compression Lap Splice, (L_c)(in.)
Bar #	**f'_c = 4.0 ksi**	**f'_c = 4.5 ksi**	**$f'_c \geq$ 4.0 ksi**
3	8.00	8.00	12.00
4	9.45	9.00	15.00
5	11.81	11.25	18.75
6	14.18	13.50	22.50
7	16.54	15.75	26.25
8	18.90	18.00	30.00
9	21.32	20.30	33.84
10	24.00	22.86	38.10
11	26.65	25.38	42.30
14	32.00	30.47	50.79
18	42.66	40.63	67.71

Notes:
1. Values based on use of normal weight concrete.
2. Values based on use of grade 60 reinforcement.
3. The minimum compression development length is 8 in.
4. The minimum compression lap splice length is 12 in.
5. Where bars of different sizes are lap spliced in compression, the splice length shall not be less than the development length of the larger bar or the splice length of the smaller bar.
6. See AASHTO 5.10.8.2.2 and 5.10.8.4.5.

Calculation Variables:

Basic Development Length, L_{db} = $0.63 * d_b * fy / sqrt(f'c)$

L_{db}(lower limit) = $0.3 * d_b * fy$

Minimum Compression Lap Splice, L_c = $m * (0.9 * f_y - 24) * d_b$

Modification Factor, m = 1.0

Table 5A-5: Tension Development Length of 90 and 180 Degree Standard Hooks

Bar #	L_{hb} (in.)	Standard Hook Tension Development Length L_{dh} (in.)			
		Cover Factor λ_{RC} = 1.0		Cover Factor λ_{RC} = 0.8	
		Spacing Factor λ_{rc} = 1.0	Spacing Factor λ_{rc} = 0.8	Spacing Factor λ_{rc} = 1.0	Spacing Factor λ_{rc} = 0.8
3	6.7	6.72	6.00	6.00	6.00
4	9.0	8.96	7.17	7.17	6.00
5	11.2	11.20	8.96	8.96	7.17
6	13.4	13.44	10.75	10.75	8.60
7	15.7	15.67	12.54	12.54	10.03
8	17.9	17.91	14.33	14.33	11.46
9	20.2	20.21	16.17	16.17	12.93
10	22.7	22.75	18.20	18.20	14.56
11	25.3	25.26	20.21	20.21	16.17

Notes:
1. Values based on use of normal weight concrete.
2. The minimum development length is 6 in.
3. See AASHTO 5.10.8.2.4.

Calculation Variables:

Basic Development Length, L_{hb} =	$38*d_b/60*fy/(\lambda*\text{sqrt}(f'c))$
L_{hb}(lower limit) =	$8*d_b$
Concrete Density Modification factor, λ	taken as 1.0
Reinforcing Steel Yield Strength, fy =	60 ksi
Compressive Strength of Concrete, f'c =	4.5 ksi

Table 5A-6: Class B Tension Lap Splice Lengths of Deformed Bars

	Class B Tension Lap Splice Length of Uncoated Deformed Bars (in.)							
	λ_{rc} = 0.4		λ_{rc} = 0.6		λ_{rc} = 0.8		λ_{rc} = 1.0	
Bar #	**Top Bars**	**Others**	**Top Bars**	**Others**	**Top Bars**	**Others**	**Top Bars**	**Others**
3	17.21	15.60	25.81	19.86	34.42	26.47	43.02	33.09
4	22.94	17.65	34.42	26.47	45.89	35.30	57.36	44.12
5	28.68	22.06	43.02	33.09	57.36	44.12	71.70	55.15
6	34.42	26.47	51.62	39.71	68.83	52.95	86.04	66.19
7	40.15	30.89	60.23	46.33	80.30	61.77	100.38	77.22
8	45.89	35.30	68.83	52.95	91.78	70.60	114.72	88.25
9	51.76	39.82	77.64	59.73	103.52	79.63	129.41	99.54
10	58.28	44.83	87.42	67.24	116.56	89.66	145.70	112.07
11	64.70	49.77	97.05	74.66	129.41	99.54	161.76	124.43

Notes:

1. Values based on use of normal weight concrete.
2. Top bars are horizontal bars placed so more than 12 in. of fresh concrete is cast below the reinforcement.
3. The minimum tension lap splice length is 12 in.
4. See AASHTO 5.10.8.4.3a.

Calculation Variables:

Class B Lap Splice Length =	1.3*L_d
Reinforcing Steel Yield Strength, fy =	60 ksi
Compressive Strength of Concrete, f'c =	4.5ksi
Reinforcement Confinement Factor, λ_{rc}:	User shall calculate

Table 5A-7: Class B Tension Lap Splice Lengths of Epoxy-Coated Bars (Coating Factor = 1.5)

	Class B Tension Lap Splice Length of Epoxy-Coated Steel Reinforcing Bars (in.) λ_{cf} = 1.5 (cover less than 3*db or clear spacing between bars less than 6*db)							
	λ_{rc} = 0.4		λ_{rc} = 0.6		λ_{rc} = 0.8		λ_{rc} = 1.0	
Bar #	Top Bars	Others	Top Bars	Others	Top Bars	Others	Top Bars	Others
3	22.50	19.86	33.75	29.78	45.01	39.71	56.26	49.64
4	30.00	26.47	45.01	39.71	60.01	52.95	75.01	66.19
5	37.50	33.09	56.26	49.64	75.01	66.19	93.76	82.73
6	45.01	39.71	67.51	59.57	90.01	79.42	112.51	99.28
7	52.51	46.33	78.76	69.49	105.01	92.66	131.27	115.82
8	60.01	52.95	90.01	79.42	120.02	105.90	150.02	132.37
9	67.69	59.73	101.53	89.59	135.38	119.45	169.22	149.31
10	76.21	67.24	114.32	100.87	152.42	134.49	190.53	168.11
11	84.61	74.66	126.92	111.99	169.22	149.31	211.53	186.64

Notes:

1. Values based on use of normal weight concrete.
2. Top bars are horizontal bars placed so more than 12 in. of fresh concrete is cast below the reinforcement.
3. The minimum tension lap splice length is 12 in.
4. λ_{rc} is the Reinforcement Confinement Factor (user shall calculate).
5. See AASHTO 5.10.8.4.3a.

Calculation Variables:

Class B Lap Splice Length = 1.3*L_d
Reinforcing Steel Yield Strength, fy = 60 ksi
Compressive Strength of Concrete, f'c = 4.5 ksi

Table 5A-8: Class B Tension Lap Splice Lengths of Epoxy-Coated Bars (Coating Factor = 1.2)

	Class B Tension Lap Splice Length of Epoxy-Coated Steel Reinforcing Bars (in.) λ_{cf} = 1.2 (cover at least 3*db and clear spacing between bars at least 6*db)							
	λ_{rc} = 0.4		λ_{rc} = 0.6		λ_{rc} = 0.8		λ_{rc} = 1.0	
Bar #	Top Bars	Others	Top Bars	Others	Top Bars	Others	Top Bars	Others
3	20.65	15.88	30.97	23.83	41.30	31.77	51.62	39.71
4	27.53	21.18	41.30	31.77	55.07	42.36	68.83	52.95
5	34.42	26.47	51.62	39.71	68.83	52.95	86.04	66.19
6	41.30	31.77	61.95	47.65	82.60	63.54	103.25	79.42
7	48.18	37.06	72.27	55.60	96.37	74.13	120.46	92.66
8	55.07	42.36	82.60	63.54	110.13	84.72	137.67	105.90
9	62.11	47.78	93.17	71.67	124.23	95.56	155.29	119.45
10	69.93	53.80	104.90	80.69	139.87	107.59	174.83	134.49
11	77.64	59.73	116.46	89.59	155.29	119.45	194.11	149.31

Notes:

1. Values based on use of normal weight concrete.
2. Top bars are horizontal bars placed so more than 12 in. of fresh concrete is cast below the reinforcement.
3. The minimum tension lap splice length is 12 in.
4. λrc is the Reinforcement Confinement Factor (user shall calculate).
5. See AASHTO 5.10.8.4.3a.

Calculation Variables:

Class B Lap Splice Length = 1.3*L_d
Reinforcing Steel Yield Strength, fy = 60 ksi
Compressive Strength of Concrete, f'c = 4.5 ksi

APPENDIX 5B - GIRDER PRELIMINARY DESIGN AIDS

General

The following table and graphs are design aids to assist with the selection of girder types and spacing for preliminary design only. See Section 6 for span capabilities of standardized steel girders.

Design assumptions for the table and the graphs are the same, except the f'ci in the table may be up to 8,500 psi at the time of post-tensioning for spliced spans.

Table 5B-1

The span capabilities shown may be limited by maximum shipping weight (see Section 5.5.1.9) or site-specific limitations. For the table, the following assumptions apply:

- No splices in simple span
- One splice in end spans
- Two splices in interior spans

Haunched pier segments were not assumed but may be feasible. Pier segments may require a thickened top flange and a thickened web. Economic spliced span capabilities were based on 4 ft. clear between flanges.

The box section properties shown are for 6 in. webs, 6 in. bottom flange, and 4 in. top flange. Actual box depths used on a project should optimize use of the available superstructure depth.

Figures 5B-1 through 5B-3

The graphs are intended to provide a quick means to compare relative costs between options. Actual cost estimates should reflect unit costs based on specific project constraints and current market conditions.

Table 5B-1: Economic Span Capabilities

PRECAST SECTION PROPERTIES							ECONOMIC SPAN CAPABILITES			
APPROXIMATE							SIMPLE SPAN		SPLICED	
NAME	WIDTH IN	AREA IN^2	CG IN	INERTIA IN^4	EMS IN	EE IN	FROM FT	TO FT	END FT	INT FT
BT84	43	948	41.7	875207	5	22	120	172	200	240
BT72	43	864	35.8	594437	5	20	106	178	180	210
BT63	43	801	31.4	425875	5	18	90	162	160	190
BT54	43	738	27	289236	5	16	72	-143	140	170
BT42	43	654	21.1	153066	5	14	55	-114	114	130
BX44	72	1128	20.5	319160	3	~9	116	133	N/A	N/A
BX44	48	906	20.7	224630	3	~12	75	128	140	170
BX35	72	1038	16.1	177917	3	~7	95	-128	N/A	N/A
BX35	48	780	16.6	129108	3	~10	65	108	110	130
BX24	72	906	11.1	68313	3	~6	-79	-88	N/A	N/A
BX24	48	666	11.3	46880	3	~7	44	-79	N/A	N/A
BX18	72	834	8.4	31885	3	~5	-65	-71	N/A	N/A
BX18	48	594	8.5	21557	3	~6	36	-65	N/A	N/A
SL16	72	1152	8	24576	2.4	2.4	41	-47	N/A	N/A
SL14	72	1008	7	16464	2.4	2.4	36	-42	N/A	N/A
SL12	72	864	6	10368	1.9	1.9	31	-40	N/A	N/A
SL10	72	720	5	6000	1.8	1.8	25	-37	N/A	N/A
SL8	72	576	4	3072	1.8	1.8	24	-31	N/A	N/A
SL6	72	432	3	1296	1.7	1.7	14	-24	N/A	N/A
SL4	72	288	2	384	1.7	1.7	0	14	N/A	N/A

\- Designates a span length which requires continuity to control liveload deflection.

N/A Designates sections that typically cannot benefit from spliced design.

~ Designates typical EE if harping is used. Path may be harped and/or sleeved strands and/or bottom slab thickening used near supports to control stresses.

EMS and EE may vary due to design requirements and shop capabilities, representative values are shown.

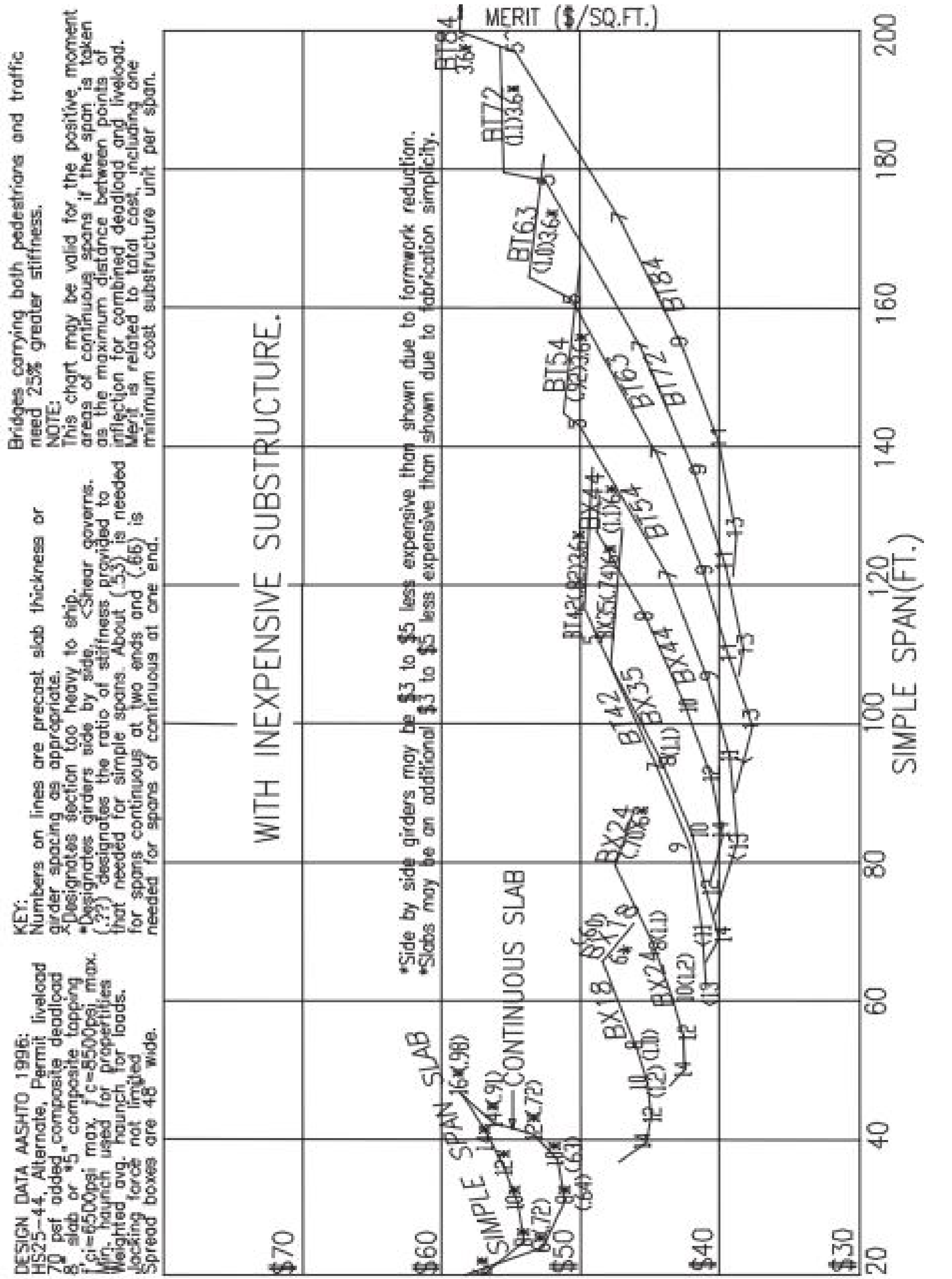


Figure 5B-1: Simple Span Girder Capabilities with Inexpensive Substructures

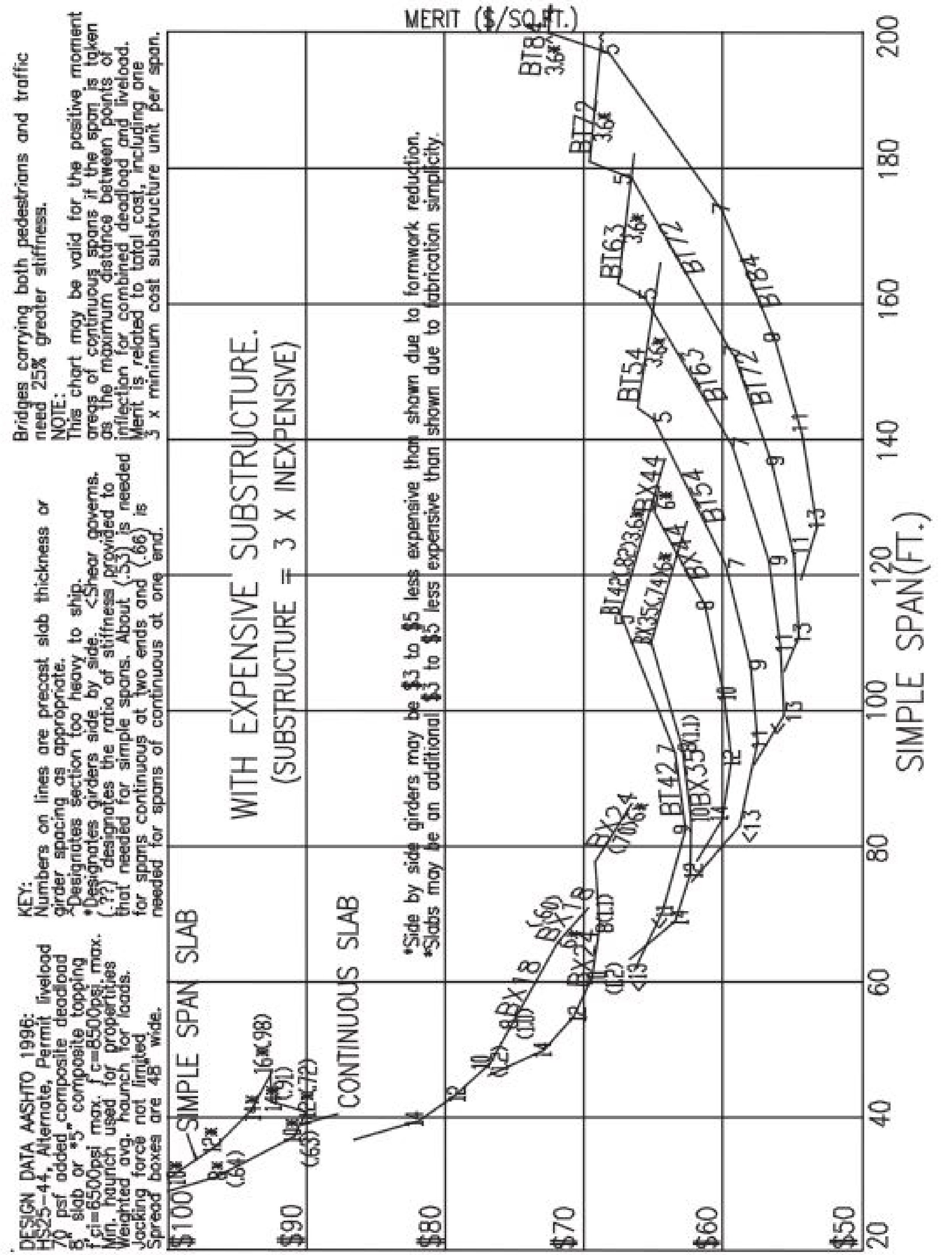


Figure 5B-2: Simple Spans Girder Capabilities with Expensive Substructures

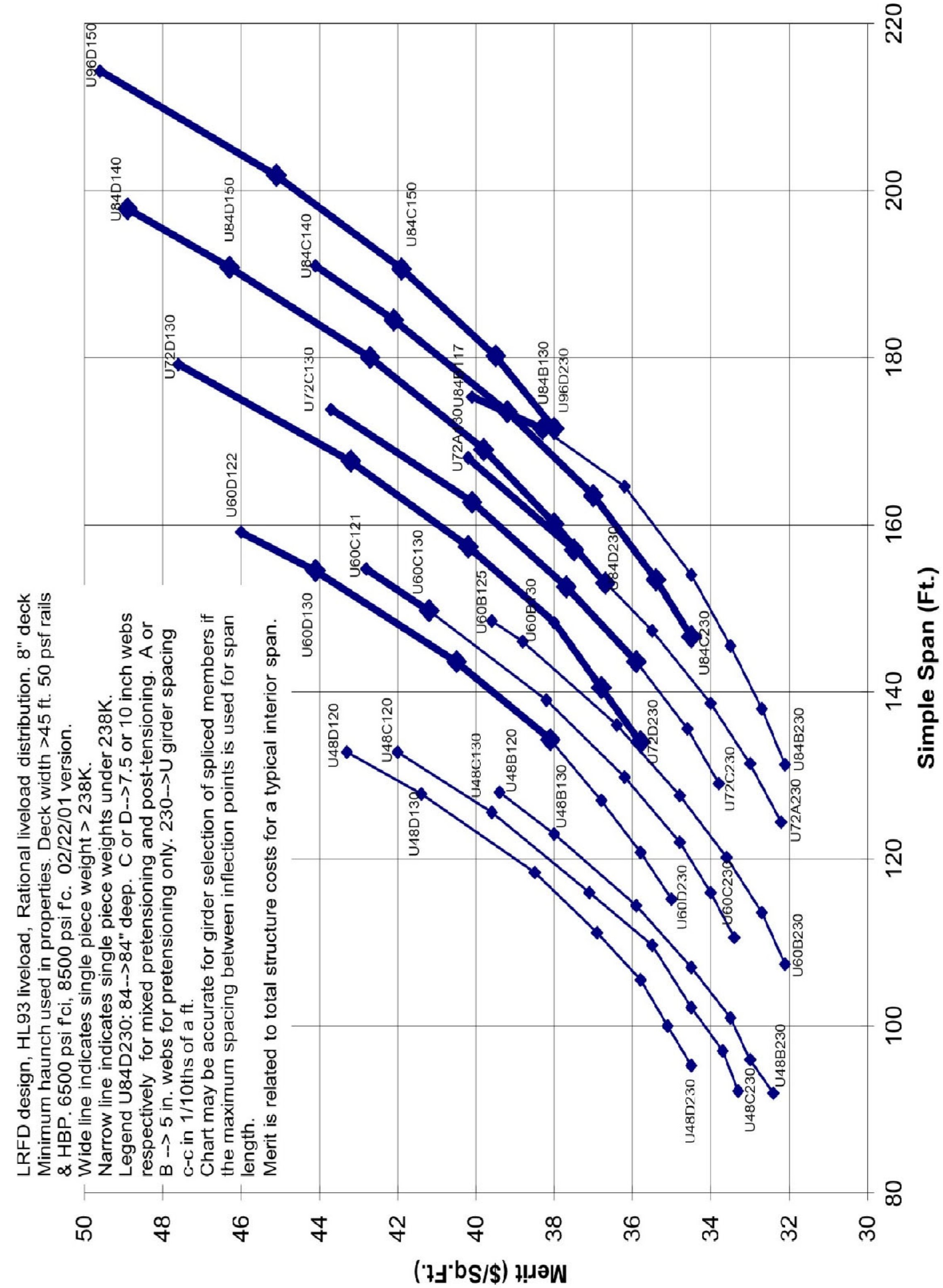
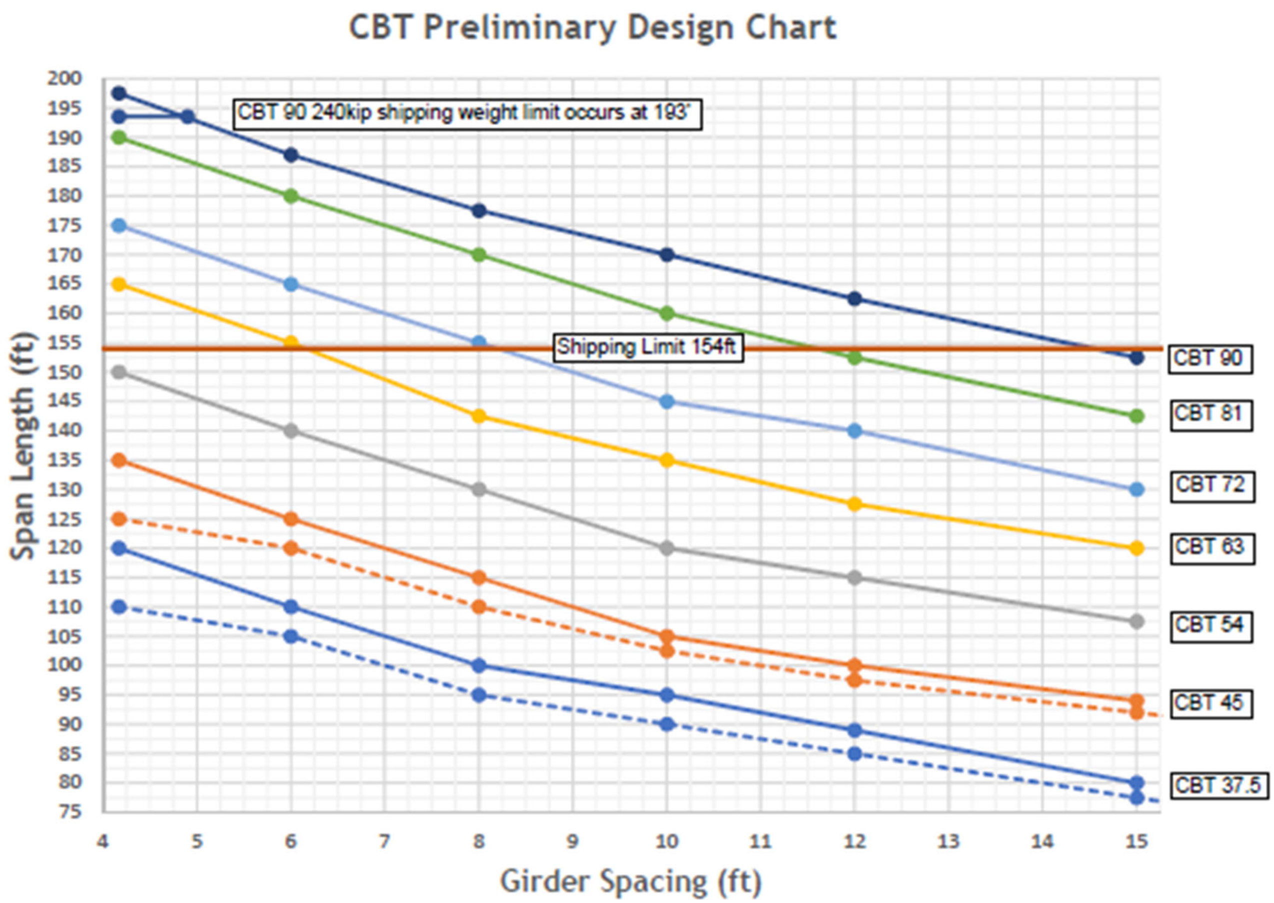


Figure 5B-3: Proposed Colorado U Girders



Notes:

Design Criteria: LRFD 9th edition, simple spans assumed with no skew. Design aid may be used for continuous spans per LRFD 5.12.3.3.

Criteria complies with CDOT BDM standard practices and policies including but not limited to concrete strengths, camber under tolerance, hold down forces, zero tension under dead load, etc.

Solid lines denote harped strands, dashed lines denote straight strands

Complies with local manufacturers' capabilities

Assumed loads:

DC Noncomposite:	4" total weighted average haunch (1" used for section properties)
DC Composite:	Type 9 bridge rail
DW Composite:	42psf between curbs for 3" HMA overlay and utilities
LL:	HL-93 & Colorado Permit Vehicle

Figure 5B-4: Colorado CBT Girders