

SECTION 6 STEEL STRUCTURES

6.1 GENERAL REQUIREMENTS

The following section is provided as CDOT practice for steel structure design. The Designer shall coordinate with Staff Bridge regarding project-specific circumstances warranting deviations from standard practices referenced herein.

This section is complementary to the CDOT Bridge Detail Manual, CDOT Standard Specifications for Road and Bridge Construction, Bridge Structural Worksheets, and AASHTO/AWS D1.5M/D1.5 Bridge Welding Code.

CDOT strongly recommends constructability reviews for new steel structures; review Section 37 of this BDM for more information.

Refer to BDM Section 10 – Foundations for steel piling design.

Refer to BDM Section 33 – Preservation and Rehabilitation of Structures for rehabilitation of steel structures.

The following recommended resources for steel bridge design include design examples of I-girder and tub/box girder design:

- American Institute of Steel Construction (AISC) / National Steel Bridge Alliance (NSBA) website (<https://www.aisc.org/nsba/>)
- FHWA website (<https://www.fhwa.dot.gov/bridge/steel.cfm>)

Found on the NSBA website, the “Short Span Steel Alliance Standards” should be used only for preliminary design and are not acceptable for final design. Refer to Section 37 of this BDM for acceptable final design calculations.

Flame or heat straightening method to repair damaged primary load carrying steel girders can only be used up to three times if the damage occurs at the same location. Replacement of the girder will be required if additional impacts are encountered.

6.2 CODE REQUIREMENTS

Designs shall be consistent with AASHTO, unless modified herein.

6.3 MATERIAL PROPERTIES

The economics of design, expected length, and location of a bridge generally govern the choice of girder material. Steel girders shall be rolled I-beams, welded plate I-girders, or tub/box girders.

6.3.1 Steel Components

AASHTO 6.4.1

All structural steel components, including structural steel, bolts, nuts, washers, and shear connectors, shall be in accordance with AASHTO steel grades, strengths, available thicknesses, and properties.

Assume 50 ksi as the default steel yield strength. CDOT allows hybrid sections. During the design phase, the Designer should contact fabricators and NSBA to verify that the design cross section is the most economical.

Generally, unpainted weathering steel may be specified for CDOT bridge girders, with the following exceptions:

1. Highway overpasses, with minimum vertical clearance, subject to a “tunnel” effect, where vertical abutments are used and full height retaining walls continue parallel to the abutment. This situation does not allow roadway spray with deicing salt to dissipate by air currents.
2. Low level water crossings where the girder has less than 8 ft. clearance to the Ordinary High Water elevation. This situation could result in prolonged periods of wetness of the steel.

When specifying unpainted weathering steel, the last 6 ft. of girders on either side of an expansion joint shall be painted, equivalent to Federal Standard 595B Color No. 30045 (weathered steel color).

Unpainted weathering steel shall not be used for railings. See Section 2.4 and Section 13 of this BDM for minimum railing and fencing steel requirements.

Refer to Section 14 of this BDM for additional information on bearing design.

6.3.1.1 Bolts

ASTM F3125 Grade A325 high strength bolts are preferred. ASTM F3125 Grade A490 bolts should be used only when necessary. It is preferred practice not to mix A325 and A490 bolt types in the same connection type. However, if the use of mixed bolt grades is justified, it is recommended that different bolt diameters be used to distinguish between the grades during construction.

A490 bolts shall not be hot-dip galvanized. If a zinc coating is required, it must follow the mechanically deposited process.

The twist-off versions of Grade A325 and A490, F1852, and F2280, respectively, are acceptable options in structural steel joints.

6.4 FATIGUE AND FRACTURE CONSIDERATIONS

AASHTO 6.6.1

6.4.1 Fatigue

Fatigue shall be categorized as load- or distortion-induced fatigue. Refer to AASHTO LRFD for fatigue design criteria for steel components and details.

6.4.2 Fracture

AASHTO 6.6.2

Refer to AASHTO LRFD for members that require mandatory Charpy V-Notch testing. If needed, Section 509 of CDOT Standard Specifications may be revised with a Project Special Provision to resolve any differences with AASHTO over which components and connections require Charpy V-Notch testing. The Designer shall clearly identify on the contract plans all components and connections requiring Charpy V-Notch testing.

The Designer shall clearly identify on contract documents all main members and/or details that are to receive non-destructive testing.

A Fracture Critical Member (FCM) is defined as a component in tension whose failure is expected to result in the collapse of the bridge or the inability of the bridge to safely carry a minimum level of traffic (live load) in its damaged condition. The Designer has the responsibility to clearly identify all FCMs on the contract plans. All FCMs shall be fabricated to AASHTO/AWS D1.5M/D1.5 Bridge Welding Code.

As a default, Staff Bridge considers the following (but not limited to) fracture critical members:

AASHTO 6.11.5

- Flanges and webs in tension on single-box girder bridges
- Flanges in tension in two-box girder bridges
- Girders or tension sub-elements in a built-up member on a bridge with fewer than three girders

The Designer has the option to perform a rigorous analysis with assumed cracked components to confirm the strength and stability of a damaged structure. However, the loading cases to be evaluated, the location of potential cracks, the appropriate level of live loads, the degree to which dynamic effects associated with a fracture are included, the refinement of models, and the choice of element type shall all be agreed upon with Staff Bridge. The ability of a software product to adequately capture the complexity of the analysis shall be considered and mutually agreed upon with Staff Bridge.

6.5 GENERAL DIMENSION AND DETAIL REQUIREMENTS**6.5.1 General**

Steel structure components shall be in accordance with the CDOT Bridge Detail Manual and AASHTO LRFD.

6.5.2 Dead Load Camber**AASHTO 6.7.2**

The general requirements for camber shall be in accordance with AASHTO LRFD, unless modified in this Section.

Steel girders should be cut or heat cambered to counteract service dead load deflections and vertical profile requirements as needed.

A tabulation showing dead load deflections for the girder only, slab only, and total shall be shown with the Girder Elevation, if “Camber and Dead Load Deflection” sheets are not used.

For straight skewed I-girder bridges and horizontally curved I-girder bridges, the Designer should clearly state in the contract documents the intended erected position of girders and fit condition. The preference is to use Steel Dead Load Fit conditions, but the Designer should consider the economic implications of using other conditions. The selected fit condition must be either recommended or acceptable in NSBA’s “Steel I-Girder Bridge Fit” Table 3 and Table 4 (shown on Figure 6-1).

Table 1 Common Fit Conditions

Loading Condition Fit	Construction Stage Fit	Description	Practice
No-Load Fit (NLF)	Fully-Cambered Fit	The cross-frames are detailed to fit to the girders in their fabricated, plumb, fully-cambered position under zero dead load.	The fabricator (detailer) sets the drops using the no-load elevations of the girders (i.e., the fully cambered girder profiles).
Steel Dead Load Fit (SDLF)	Erected Fit	The cross-frames are detailed to fit to the girders in their ideally plumb as-deflected positions under the bridge steel dead load at the completion of the erection.	The fabricator (detailer) sets the drops using the girder vertical elevations at steel dead load, calculated as the fully cambered girder profiles minus the steel dead load deflections.
Total Dead Load Fit (TDLF)	Final Fit	The cross-frames are detailed to fit to the girders in their ideally plumb as-deflected positions under the bridge total dead load.	The fabricator (detailer) sets the drops using the girder vertical elevations at total dead load, which are equal to the fully cambered girder profiles minus the total dead load deflections.

Table 3 Recommended Fit Conditions for Straight I-Girder Bridges
 (including Curved I-Girder Bridges with L/R in all spans ≤ 0.03)¹

Square Bridges and Skewed Bridges up to 20 deg Skew			
	Recommended	Acceptable	Avoid
Any span length	Any		None
Skewed Bridges with Skew > 20 deg and $I_s \leq 0.30$ +/-			
	Recommended	Acceptable	Avoid
Any span length	TDLF or SDLF		NLF
Skewed Bridges with Skew > 20 deg and $I_s > 0.30$ +/-			
	Recommended	Acceptable	Avoid
Span lengths up to 200 ft +/-	SDLF	TDLF	NLF
Span lengths greater than 200 ft +/-	SDLF		TDLF & NLF

Table 4 Recommended Fit Conditions for Horizontally Curved I-Girder Bridges
 ($(L/R)_{MAX} > 0.03$)¹

Radial or Skewed Supports			
	Recommended	Acceptable	Avoid
$(L/R)_{MAX} \geq 0.2$	NLF ^{2,3}	SDLF ⁴	TDLF
All other cases	SDLF	NLF	TDLF

Note 1: For the various recommended fit conditions presented in Tables 3 and 4, the span length and skew index limits should be considered approximate guidelines and should be evaluated in the full context of the geometric and structural complexity of the given bridge.

Note 2: The recommendation transitions to NLF at or above a maximum L/R of 0.2 because research on these types of bridges (NCHRP 2015) shows that the increase in the cross-frame forces from SDLF detailing can become more significant as the degree of curvature increases. NLF matches the normal analysis methods used in the design and will provide a better match between predicted forces and displacements than SDLF when the steel dead load displacements become large.

Note 3: The recommendation to use NLF detailing does not necessarily imply the need to use temporary shoring, nor does the use of SDLF or TDLF detailing imply that temporary shoring cannot be used. The choice of detailing method affects the nature and magnitude of the bridge's internal dead load forces as well as the "fit-up" forces which the erector may need to apply to assemble the structural steel. The nature and magnitude of these forces are also influenced by the use of temporary shoring. Bridges erected without temporary shoring can be detailed for NLF and successfully erected if the fit-up forces are manageable. Likewise, bridges which are to be erected using some form of temporary shoring can be detailed for SDLF or TDLF and successfully erected if the fit-up forces are manageable.

Note 4: SLDF detailing is considered acceptable in these cases if the additive locked-in force effects are considered

L = span length, bearing to bearing along the centerline of the bridge

R = radius of the centerline of the bridge cross-section

I_s = skew index

Figure 6-1: Recommended Fit Conditions

The complete document is found on NSBA's website; the link is provided at the beginning of this section.

Because box girders are inherently stiff in torsion, it is difficult to achieve fit-up of Total Dead Load Fit conditions. As a result, external cross-frames are typically detailed and fabricated to fit to the girder geometry under No-Load Fit or Steel Dead Load Fit conditions, depending on the intended erection sequencing.

For curved or skewed box girder bridges where a line girder analysis was not used, report deflections along individual webs, not along the centerline of the girder.

AASHTO 6.7.3

6.5.3 Minimum Thickness of Steel

Refer to AASHTO LRFD and AASHTO/NSBA G12.1, *Guidelines to Design for Constructability*, for minimum thicknesses of steel elements.

AASHTO 6.7.4

6.5.4 Diaphragms and Cross-Frames

Refer to AASHTO LRFD for the design and placement of diaphragms or cross-frames for main I-beam and box section members.

CDOT prefers bolted connections. No tack welding is allowed before bolting.

Per AASHTO, all members included in the structural model that are used to determine girder force effects shall be designed as primary members. This includes all diaphragms or cross-frames in horizontally curved and heavily skewed bridges.

In general, for bridges with skew angles of 20° or less, the diaphragms or cross-frames shall be placed parallel to the centerline of the support.

For bridges with skew angles greater than 20°, the diaphragms or cross-frames shall be placed perpendicular to the main members.

Where a support line at an interior pier is skewed more than 20° from normal, elimination of the diaphragms or cross-frames along the skewed interior support line may be considered. Verify with Staff Bridge that this is an acceptable option.

6.5.4.1 Box Section Members

The need for temporary or permanent intermediate internal diaphragms or cross-frames, external diaphragms or cross-frames, top lateral bracing, or other means shall be investigated to ensure that deformations of the box section are controlled. If temporary intermediate diaphragms are specified, they shall be removed once the entire deck is poured and has achieved its full design strength.

6.6 I-SECTION FLEXURAL MEMBERS

Refer to Section 9.4.2 of this BDM for design of concrete decks and overhangs.

CDOT does not allow chorded girders, except for a simple made continuous design (see Section 6.6.6).

AASHTO 6.10.1.1b
6.10.1.1c

6.6.1 Composite Sections

The Designer shall refer to AASHTO LRFD to calculate composite section properties in positive and negative moment regions.

In situations where AASHTO does not consider the concrete deck when calculating composite girder section properties in negative moment regions, only longitudinal reinforcing in the top mat, within the effective deck width, shall be considered effective. If a project does not allow precast deck panels, then the bottom longitudinal reinforcing may also be considered effective when calculating said section properties.

AASHTO 6.10.1.7

6.6.2 Minimum Negative Flexure Concrete Deck Reinforcement

Refer to AASHTO LRFD for the minimum negative moment flexure reinforcement.

6.6.3 Non-composite Sections

AASHTO 6.10.10.3

CDOT does not permit the use of non-composite sections in positive moment locations. If the Designer finds that it is not economical to use composite sections in negative moment regions, shear connectors and longitudinal reinforcing shall satisfy requirements in AASHTO LRFD for the contra-flexure points.

6.6.4 Constructability

AASHTO 6.10.3

Satisfy all requirements in AASHTO LRFD for primary members at all critical construction phases.

6.6.4.1 Wind Loads During Construction

The Designer shall verify stability of girders due to wind loading on girders during construction. Be aware that not all design software addresses these checks. Refer to *AASHTO Guide Specifications for Wind Loads on Bridges During Construction* for guidance.

6.6.4.2 Deck Placement

AASHTO 6.10.3.4

Sections in positive flexure that are composite in the final condition, but are non-composite during construction, shall be investigated for flexure during the various stages of deck placement.

Cantilevered brackets placed along the exterior girders typically support concrete deck overhang construction loads. The overhang brackets with construction loads, such as screeds, can result in excessive deflections and rotation on exterior girders. The Designer may assume a deck overhang bracket configuration shown in Figure 6-2, with the brackets extending to the bottom flange, which is preferred. Alternatively, the brackets may bear on the girder web if means are provided to ensure that the web is not damaged and the associated deformations permit proper placement of the concrete deck.

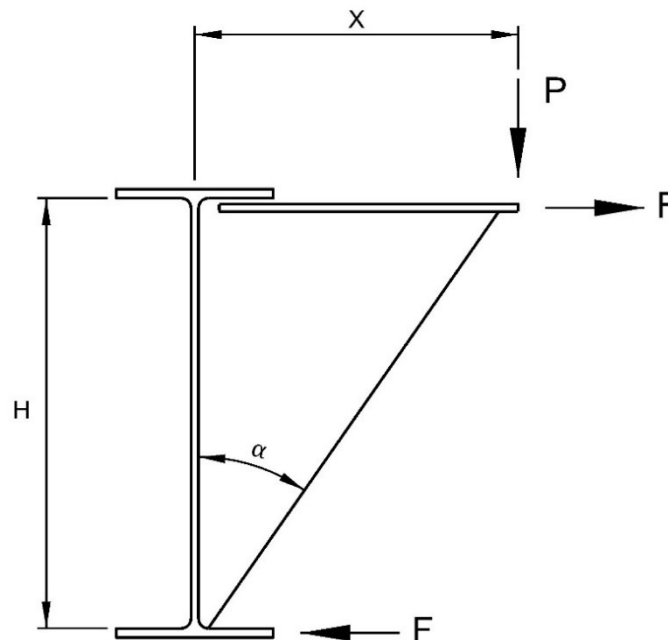


Figure 6-2: Deck Overhang Bracket

Although the brackets are typically spaced at 3 to 4 ft. along the exterior girder, all bracket loads except the finishing machine load are assumed to be applied uniformly. The Designer calculates the vertical load, P , acting at the edge of the overhang bracket. The bracket is assumed to extend near the edge of the deck overhang; therefore, half the deck overhang weight is placed on the exterior girder and half the weight is placed on the overhang brackets. Designers may conservatively include one-half the deck haunch weight in the total overhang weight.

Construction loads or dead loads and temporary loads that act on the overhang only during construction are assumed (as minimum) as follows:

Overhang deck forms: $P = 40 \text{ lbs/ft.}$

Screed rail: $P = 85 \text{ lbs/ft.}$

Railing: $P = 25 \text{ lbs/ft.}$

Walkway: $P = 125 \text{ lbs/ft.}$

Finishing machine: $P = 3,000 \text{ lbs}$

The finishing machine load is estimated as one-half of the total finishing machine truss weight, plus additional load to account for the weight of the engine; drum and operator are assumed to be located on one side of the truss. Note: The above loads are estimates only. It is recommended that the Designer contact the Contractor, if known at the time of design, to obtain more accurate construction load values. Otherwise, the Designer shall validate the assumed loads during review of construction submittals regarding the deck forming system and finishing machine.

Falsework shall not be used for new construction and deck replacement construction. If falsework appears necessary during design, discuss with Staff Bridge.

6.6.5 Longitudinal Stiffeners and Cover Plates

CDOT does not allow longitudinal stiffeners and cover plates on new construction without Unit Leader approval in coordination with the State Bridge Engineer.

6.6.6 Simple Made Continuous

The Designer is encouraged to consider simple made continuous (SMC) bridges in the design of multi-span structures. SMC bridges reduce uplift in unbalanced spans, reduce negative moments at the piers, simplify fabrication, and eliminate the need for bolted field splices.

Critical to the functionality of SMC structures is the continuity connection at the piers. As industry best practices develop with new research, the Designer is encouraged to discuss with Staff Bridge to select the most appropriate connection details for design and construction.

AISC *Engineering Journal*, Second Quarter, 2014, provides commentary on several connection details and a design procedure for SMC bridges.

6.7 TUB/BOX-SECTION FLEXURAL MEMBERS

6.7.1 General

This section supplements AASHTO Section 6.11.

For reference, see the NSBA website for design examples for straight and curved box/tub girders (<https://www.aisc.org/nsba/nsba-publications/steel-bridge-design-handbook/>).

This section also covers press brake tub girders (PBTG). For more details, see Appendix 6C. NSBA will be providing a Press Brake Tub Girder Design Manual.

6.7.2 Bearings

Straight, not skewed, tub girders may use two bearings at supports. Curved or skewed tub girders should have one bearing at supports.

6.7.3 Cross-Section Proportion Limits

In 2006, the Transportation Research Board (TRB) published a paper "Practical Steel Tub Girder Design," providing guidance for preliminary design considerations, including preliminary girder sizing and spacing.

Bottom flange longitudinal stiffeners are permitted, but unstiffened bottom flanges are preferred. Using longitudinal stiffeners may result in undesirable fatigue details. The Designer should investigate thickening the bottom flange and/or reducing the bottom flange width in lieu of using longitudinal stiffeners. Bottom longitudinal stiffeners that terminate at the girder ends or at the end of the flange at splices do not present a fatigue problem.

Box girder segment widths greater than 12 ft. may present transportation issues and should be avoided where feasible. Be aware of girder curvature because it increases the overall out-to-out segment width. Consult with fabricators on shipping when wider segments could eliminate an extra field splice.

Maximum allowable shipping lengths are highly variable from state to state, but 120 ft. is a common restriction. Splices may be used, but the Designer is recommended to consult potential fabricators of tub/box girders when laying out field splices.

Provisions for adequate draining and ventilation of the interior of the tub are essential. Appendix 6A provides a typical drain hole detail.

6.8 CONNECTIONS AND SPLICES

6.8.1 Bolted Connections

AASHTO 6.13.2.1.1

Bolted connections of primary members subjected to significant load reversals, heavy impact loads, severe vibration or where a joint slippage would be detrimental to the serviceability of the structure shall be designated and designed as slip-critical. Load cases during construction should be considered.

Typical slip-critical connections include, but are not limited to, the following connections:

- Girder splices
- Connections for primary member diaphragm that experience axial tension or combined axial tension and shear
- Any connection in shear with oversized or slotted holes

The most typical surface condition used in Colorado is Class A, unpainted clean mill scale, and blast-cleaned surfaces with Class A coatings. Where special consideration is necessary, Class B surfaces may be submitted to Staff Bridge for approval by Unit Leader in coordination with the Fabrication/Construction Unit. When Class B friction surfaces are used, the plans shall specify the connection surface conditions that must be present at the time of bolting.

6.8.2 Flange Splices

CDOT allows flange width and thickness transitions at splices. The Designer should ensure that enough material is saved for the flange transition to be cost effective.

6.8.3 Welded Connections

Other than welds between girders and bearing plates, CDOT does not permit any field welds or permanent tack welds.

Full penetration welds on webs and flanges made with backing should not be allowed. The following pre-qualified welds may be used: B-U3c-S, B-L1a-S, B-L2c-S, B-U6-S, C-U6-S, and B-U7-S.

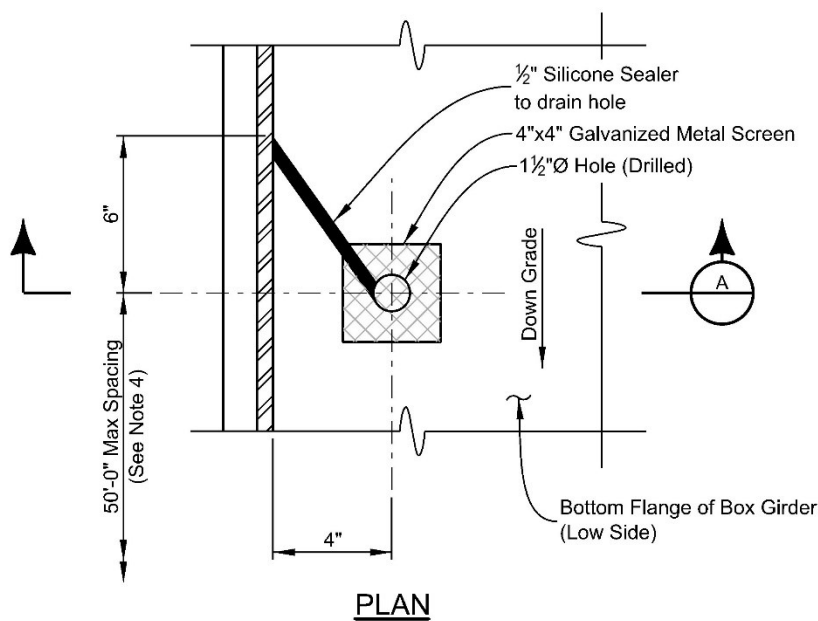
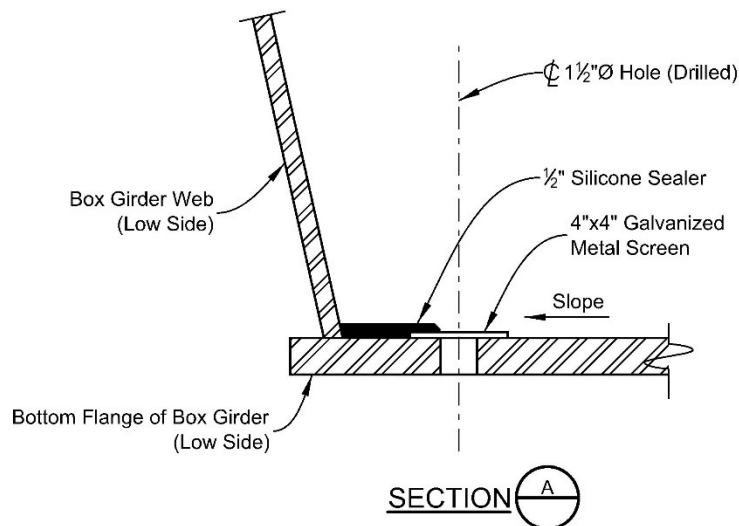
6.9 REFERENCES

Azizinamini, Atorod. "Simple for Dead Load-Continuous for Live Load Steel Bridge Systems." *Engineering Journal*. American Institute of Steel Construction, Vol. 51, Quarter 2 (2014): 59–82.

Coletti, Domenic; Zhanfei (Tom) Fan; John Holt; John Vogel. "Practical Steel Tub Girder Design." Transportation Research Board 85th Annual Meeting, 2006.

Robert I. Johnson, and Rebecca A. Atadero. "Simple-Made-Continuous Steel Bridges with Steel Diaphragms." *Engineering Journal*. American Institute of Steel Construction, Vol. 54, Quarter 1 (2017): 3–20.

APPENDIX 6A - TYPICAL TUB/BOX GIRDER DETAILS



GIRDER DRAIN HOLE DETAILS

NOTES:

1. Clean steel before applying silicone sealer compound.
2. Compound is subject to approval of Engineer.
3. Silicone sealer will not be paid for separately, but included in the cost of the work.
4. In addition, provide drain holes at low points of box girder.
5. Galvanized metal screen shall be included in drain hole cost. Attach to top of bottom flange using Engineer approved adhesive.



APPENDIX 6B - 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.

A simple spreadsheet was developed by CSU in conjunction with a research project entitled “Development of Steel Design Details and Selection Criteria for Cost-Effective and Innovative Steel Bridges in Colorado”, Report No. CDOT-2008-12. The design tables below are a small sample of the design tables produced. The tables shown below are for a 44 foot width bridge and simple spans. Additional tables are available in the report and the spreadsheet is available on the CDOT website. These tables and the software is based on rolled beam shapes and simple for dead load and continuous for live load details. For continuous steel girders and longer span capabilities, additional design standards can be found at: <https://www.aisc.org/nsba/design-resources/continuous-span-standards/>

Some Design Assumptions:

- 8 - 9" slab depending on girder spacing 4.5 ksi concrete w/ 4" future wearing surface
- 2 – 2.5 ft Overhang
- C15 x 33.9 Diaphragms
- 18 ft interior and 12 ft exterior diaphragm spacing
- 3 rows of 5" x 7/8" Shear Studs spaced at 5.25" or 6" dia throughout length for conservative estimate
- 2 – 486 lbs/ft barriers with 1.5 ft width
- 2 design lanes when out to out width was 44 ft or less, 3 design lanes for widths greater than 44 ft
- Weight estimate per square foot includes: Lightest wide flange beam weight, shear studs, and diaphragm weight
- All design charts were designed using a HL-93 Design Truck

60 ft span

Longest Span	L	60	ft	Nominal Depth		Weight per linear foot	Erected Cost per Square Foot (Steel)	Pounds per Square Foot (Steel)
Full Width	w	44	ft	W40	X	167	\$17.17	19.05
Slab Thickness	Ts	9	in	W36	X	170	\$17.41	19.32
No. of girders	Nb	4		W36	X	182	\$18.36	20.42
Girder spacing	S	13	ft	W40	X	183	\$18.43	20.51
Overhang		2.5	ft	W30	X	191	\$19.06	21.23

Longest Span	L	60	ft	Nominal Depth		Weight per linear foot	Erected Cost per Square Foot (Steel)	Pounds per Square Foot (Steel)
Full Width	w	44	ft	W33	X	141	\$18.22	19.69
Slab Thickness	Ts	8.25	in	W40	X	149	\$19.04	20.60
No. of girders	Nb	5		W36	X	150	\$19.14	20.71
Girder spacing	S	9.75	ft	W33	X	152	\$19.34	20.94
Overhang		2.5	ft	W36	X	160	\$20.15	21.85

Longest Span	L	60	ft	Nominal Depth		Weight per linear foot	Erected Cost per Square Foot (Steel)	Pounds per Square Foot (Steel)
Full Width	w	44	ft	W33	X	130	\$20.09	21.33
Slab Thickness	Ts	8	in	W30	X	132	\$20.33	21.60
No. of girders	Nb	6		W36	X	135	\$20.71	22.01
Girder spacing	S	7.8	ft	W33	X	141	\$21.45	22.83
Overhang		2.5	ft	W27	X	146	\$22.06	23.51

70 ft span

Longest Span	L	70	ft	Nominal Depth		Weight per linear foot	Erected Cost per Square Foot (Steel)	Pounds per Square Foot (Steel)
Full Width	w	44	ft	W40	X	211	\$20.66	23.15
Slab Thickness	Ts	9	in	W40	X	215	\$20.97	23.51
No. of girders	Nb	4		W36	X	231	\$22.17	24.97
Girder spacing	S	13	ft	W36	X	232	\$22.25	25.06
Overhang		2.5	ft	W40	X	235	\$22.47	25.33

Longest Span	L	70	ft	Nominal Depth		Weight per linear foot	Erected Cost per Square Foot (Steel)	Pounds per Square Foot (Steel)
Full Width	w	44	ft	W40	X	167	\$20.92	22.75
Slab Thickness	Ts	8.25	in	W36	X	182	\$22.41	24.46
No. of girders	Nb	5		W40	X	183	\$22.51	24.57
Girder spacing	S	9.75	ft	W36	X	194	\$23.58	25.82
Overhang		2.5	ft	W40	X	199	\$24.07	26.39

Longest Span	L	70	ft	Nominal Depth		Weight per linear foot	Erected Cost per Square Foot (Steel)	Pounds per Square Foot (Steel)
Full Width	w	44	ft	W40	X	149	\$22.51	24.03
Slab Thickness	Ts	8	in	W36	X	150	\$22.63	24.17
No. of girders	Nb	6		W36	X	160	\$23.84	25.53
Girder spacing	S	7.8	ft	W40	X	167	\$24.68	26.49
Overhang		2.5	ft	W33	X	169	\$24.92	26.76

80 ft span

Longest Span	L	80	ft	Nominal Depth		Weight per linear foot	Erected Cost per Square Foot (Steel)	Pounds per Square Foot (Steel)
Full Width	w	44	ft	W40	X	277	\$25.40	28.97
Slab Thickness	Ts	9	in	W40	X	278	\$25.47	29.06
No. of girders	Nb	4		W36	X	282	\$25.75	29.42
Girder spacing	S	13	ft	W33	X	291	\$26.38	30.24
Overhang		2.5	ft	W40	X	294	\$26.59	30.51

Longest Span	L	80	ft	Nominal Depth		Weight per linear foot	Erected Cost per Square Foot (Steel)	Pounds per Square Foot (Steel)
Full Width	w	44	ft	W40	X	211	\$25.08	27.55
Slab Thickness	Ts	8.25	in	W40	X	215	\$25.46	28.01
No. of girders	Nb	5		W36	X	231	\$26.96	29.83
Girder spacing	S	9.75	ft	W36	X	232	\$27.05	29.94
Overhang		2.5	ft	W40	X	235	\$27.33	30.28

Longest Span	L	80	ft	Nominal Depth		Weight per linear foot	Erected Cost per Square Foot (Steel)	Pounds per Square Foot (Steel)
Full Width	w	44	ft	W40	X	183	\$26.43	28.46
Slab Thickness	Ts	8	in	W36	X	194	\$27.71	29.96
No. of girders	Nb	6		W40	X	199	\$28.29	30.64
Girder spacing	S	7.8	ft	W33	X	201	\$28.53	30.91
Overhang		2.5	ft	W36	X	210	\$29.56	32.14

90 ft span

Longest Span	L	90	ft	Nominal Depth		Weight per linear foot	Erected Cost per Square Foot (Steel)	Pounds per Square Foot (Steel)
Full Width	w	44	ft	W40	X	324	\$28.69	33.32
Slab Thickness	Ts	9	in	W36	X	361	\$31.12	36.69
No. of girders	Nb	4		W40	X	362	\$31.18	36.78
Girder spacing	S	13	ft	W40	X	372	\$31.81	37.69
Overhang		2.5	ft	W33	X	387	\$32.75	39.05

Longest Span	L	90	ft	Nominal Depth		Weight per linear foot	Erected Cost per Square Foot (Steel)	Pounds per Square Foot (Steel)
Full Width	w	44	ft	W40	X	249	\$28.69	31.96
Slab Thickness	Ts	8.25	in	W40	X	264	\$30.05	33.67
No. of girders	Nb	5		W40	X	277	\$31.21	35.15
Girder spacing	S	9.75	ft	W40	X	278	\$31.30	35.26
Overhang		2.5	ft	W36	X	282	\$31.65	35.71

Longest Span	L	90	ft	Nominal Depth		Weight per linear foot	Erected Cost per Square Foot (Steel)	Pounds per Square Foot (Steel)
Full Width	w	44	ft	W40	X	215	\$30.21	32.92
Slab Thickness	Ts	8	in	W36	X	231	\$32.01	35.10
No. of girders	Nb	6		W36	X	232	\$32.12	35.24
Girder spacing	S	7.8	ft	W40	X	235	\$32.46	35.65
Overhang		2.5	ft	W33	X	241	\$33.12	36.47

APPENDIX 6C– PRESS BRAKE TUB GIRDER PRELIMINARY DESIGN AIDS

The table below shows the current span capabilities of the “Press Brake Tub Girders”. Based on a 2022 steel estimate of \$2.50/pound the girder costs will vary from \$242 to \$477 per linear foot depending on girder type (U12 to U33). Actual cost estimates should reflect unit costs based on specific project constraints and current market conditions. In general, these tub girders are considered compact sections unless skew is involved. Additional design will be required.

U-BEAM™ Selection Chart Per AASHTO LRFD Requirements

Spacing	Bridge Length (ft)															
	25	30	35	40	45	50	55	60	65	70	75	80	85	90	95	100
4' - 6"	U12	U12	U12	U12	U12	U18	U18	U18	U24	U24	U30	U30	U33	U33	S.D.	S.D.
5' - 0"	U12	U12	U12	U12	U12	U18	U18	U24	U24	U24	U30	U30	U33	S.D.	S.D.	S.D.
5' - 6"	U12	U12	U12	U12	U12	U18	U18	U24	U24	U30	U30	U33	S.D.	S.D.	S.D.	S.D.
6' - 0"	U12	U12	U12	U12	U18	U18	U18	U24	U24	U30	U30	U33	S.D.	S.D.	S.D.	S.D.
6' - 6"	U12	U12	U12	U12	U18	U18	U24	U24	U30	U30	U33	S.D.	S.D.	S.D.	S.D.	S.D.
7' - 0"	U12	U12	U12	U12	U18	U18	U24	U24	U30	U33	S.D.	S.D.	S.D.	S.D.	S.D.	S.D.
7' - 6"	U12	U12	U12	U12	U18	U24	U24	U30	U30	U33	S.D.	S.D.	S.D.	S.D.	S.D.	S.D.
8' - 0"	U12	U12	U12	U18	U18	U24	U24	U30	U33	S.D.	S.D.	S.D.	S.D.	S.D.	S.D.	S.D.