NCHRP 12-103

Two-Span Continuous Closed Steel Box Girder Bridge Design Summary

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General

* This document outlines the procedures and methods used to model, analyze, evaluate, and design a two-span continuous closed steel box girder bridge in accordance with the *AASHTO LRFD Bridge Design Specifications* (2014). Other design aids include *Practical Steel Tub Girder Design* by Colleti, Fan, Holt, and Vogel (2005); and *Design Example 4: Three-Span Continuous Straight Composite Steel Tub Girder Bridge* published by the *FHWA* and completed by *HDR Engineering* in November of 2012.

Limit States to be Satisfied:

* Strength I
* Service II
* Any additional limit states, as required

Design Parameters

* Span length

- Typical span lengths for closed steel box girder bridges range from 150 feet to 500 feet.

- Since the model used for NCHRP 12-103 determined that shorter spans control design of closed steel box girder bridges, a practical minimum span length of 150 feet is recommended.

- For the design of this two-span continuous bridge each span equals 150 feet.

* Width

- The width of the bridge is determined based on the number and width of traffic lanes provided, the width of shoulders, and width of other features, such as pedestrian sidewalks, bicycle lanes, or observation overhangs.

- The bridge cross-section consists of four rectangular steel girders with top flanges spaced at 4’-7 ¼” on center, 5’-6 ¼” between the centerline of adjacent top flanges, and 4’ overhangs for a deck width of 43.0’, out-to-out.

* Skew

- Skew is not considered in this design, but may be investigated in future work.

* Depth

- The minimum recommended depth of tub girders is 5 ft., or L/25. For this design, the depth of the section equals 5 ft.

* Width and Thickness of Overhang  
  - If empirical live load distribution factors are to be employed, the final cross-section must meet the requirements of Article 6.11.2.3, which states that the deck overhang should not exceed 60 percent of the distance between centers of the top flanges of adjacent box girders, or 6.0 feet. Also, the distance center-to-center of adjacent box girders shall not be greater than 120 percent, nor less than 80 percent, of the top flange center-to-center distance of a single box girder.
* Deck Parapets  
  - Deck parapets are each assumed to weigh 520 pounds per linear foot, and rest on the outer edge of the roadway, parallel to the direction of traffic.
* Web Thickness  
  - The webs of the girders were proportioned such that no longitudinal stiffeners were required, based on AASHTO Eq. 6.11.2.1.2-1:
* Top Flange Width  
  - The minimum width of the top flange was determined by AASHTO Eq. 6.11.2.2-2:

- However, Article C6.10.3.4 suggested additional criteria to determine flange width, where *L* is the length of the girder:

* Flange Thickness  
  - The minimum flange thickness guideline adheres to AASHTO Eq. 6.11.2.2-3:

- However, AASHTO/NSBA Steel Bridge Collaboration *Guidelines for Design and Constructibility* recommend a minimum flange thickness of 0.75 inches to enhance girder stability during handling and erection.

- The top flange must also satisfy AASHTO Eq. 6.11.2.2-1:

* Deck Thickness  
  - The total thickness of the cast-in-place concrete deck is 9.5 inches, including a 0.5-inch thick integral wearing surface.
* Concrete Deck Haunches  
  - The concrete deck haunch is 1-⅛ inches thick. The total haunch thickness, including the top flange, measures 3.5 inches.
* The steel cross-section properties are summarized in the table below:

Table 1: Steel Cross-Section Properties

|  |  |  |  |
| --- | --- | --- | --- |
| Girder | Top Flange | bf (in.) | 48.000 |
| tf (in.) | 2.000 |
| Fyc (ksi) | 50.000 |
| Web | D (in.) | 60.000 |
| tw (in.) | 0.5000 |
| Fyw (ksi) | 50.000 |
| Bottom Flange | bf (in.) | 48.000 |
| tf (in.) | 2.000 |
| Fyt (ksi) | 50.000 |
| Hybrid Girder Factor | Rh | 1.000 |
| Steel Modulus | Es (ksi) | 29000.000 |

* The slab and haunch properties are summarized in the table below:

Table 2: Slab and Haunch Properties

|  |  |  |
| --- | --- | --- |
| Slab | Structural Thickness (in.) | 9.000 |
| wc (lb/ft3) | 150.000 |
| f'c (ksi) | 4.000 |
| Ec (ksi) | 3834.254 |
| Haunch | Thickness of Haunch (in.) | 3.500 |

* The section properties for the non-composite, *n* composite, and *3n* composite sections are tabulated below:

Table 3: Section Properties for Non-composite, Short-term Composite, and Long-term Composite Sections

|  |  |  |  |
| --- | --- | --- | --- |
| *Component* | *Ix (in.4)* | *Stop (in.3)* | *Sbottom (in.3)* |
| *Girder Only* | 184576.000 | 5768.000 | 5768.000 |
| *Composite n* | 424213.297 | 16690.26 | 6734.81 |
| *Composite 3n* | 245719.043 | 9567.32 | 6412.82 |

* The bridge properties are summarized in the table below. Note that the haunch width is cast to match the width of the top flanges.

Table 4: Bridge Properties

|  |  |  |
| --- | --- | --- |
| Bridge Properties | | |
| Number of Girders | 4 | |
| Top Flange Spacing | 4.00 | ft |
| Adjacent Flange Spacing | 6.330 | ft |
| Overhang Length | 4.00 | ft |
| Roadway Width | 40.00 | ft |
| Barrier Width | 1.50 | ft |
| Out-to-out Width | 43.00 | ft |
| Deck Thickness | 9.00 | in |
| FWS Thickness | 0.50 | in |
| Total Deck Thickness | 9.50 | in |
| Haunch Thickness | 3.50 | in |
| Haunch Width | 48.000 | in |

* The weight of bridge components is tabulated below:

Table 5: Weight of Bridge Components

|  |  |  |
| --- | --- | --- |
| Weight of Components | | |
| Weight of Parapets | 0.520 | klf |
| Weight of FWS | 0.025 | ksf |
| Weight of Conc. Deck | 0.150 | kcf |
| Weight of SIP Forms | 0.015 | kcf |
| Weight of Steel | 0.490 | kcf |
| Weight of Cross Frames & Details | 0.110 | k/ft |

Select Resistance Factors – Strength I Limit State

* Applicable resistance factors were determined using AASHTO Article 6.5.4.2.

Select Load Modifiers

* The applicable load modification factors were determined based on the guidelines of AASHTO Articles 1.3.3 through 1.3.5. Table 7 displays the modification factors used in this design.

Table 6: Load Modification Factors

|  |  |
| --- | --- |
| Modification Factors | |
| Ductility | 1.0 |
| Redundancy | 1.0 |
| Operational Importance | 1.0 |

Select Applicable Load Combinations and Load Factors

* The applicable load combinations and load factors were determined using AASHTO Table 3.4.1-1 and 3.4.1-2.
* γDC = 1.25
* γDW = 1.50
* γLL = 1.75

Calculate Force Effects

* For the previously defined cross-section, *IES Visual Analysis* software determined magnitude and location of the unfactored moments. The dead loads, construction live load, and results from the HL-93 vehicular design load are shown below.

Table 7: Dead Loads

|  |  |  |  |
| --- | --- | --- | --- |
| Dead Loads | | | |
| DC1 | Deck | 5.106 | k/ft |
| Overhang Tapers | 1.150 | k/ft |
| Haunches | 0.150 | k/ft |
| SIP Forms | 0.095 | k/ft |
| Girder Self-Weight | 10.476 | k/ft |
| Cross Frames & Details | 0.110 | k/ft |
| DC1 Total (per 4 Girders) | | 17.087 | k/ft |
| DC1 Load per 1 Girder | | 4.272 | k/ft |
| DC2 | Barrier | 1.040 | k/ft |
| DC2 per 1 Girder | | 0.260 | k/ft |
| DW | FWS | 1.000 | k/ft |
| DW per 1 Girder | | 0.250 | k/ft |

Table 8: Construction Live Loads

|  |  |  |  |
| --- | --- | --- | --- |
| Live Load | | | |
| *CLL* | Construction LL | 0.430 | k/ft |
| Construction LL per Girder | 0.108 | k/ft |

Table 9: Unfactored Dead Load Moments

|  |  |  |  |
| --- | --- | --- | --- |
| Dead Load | | | |
| *DC1* | | | |
| Moment | 60 ft | 6757.00 | ft-k |
| Moment | 150 ft | -12015.00 | ft-k |
| Moment | 240 ft | -6757.00 | ft-k |
| Shear | 150 ft | 400.50 | k |
| Shear | 150 ft | -400.50 | k |
| *DC2* | | | |
| Moment | 60 ft | 411.30 | ft-k |
| Moment | 150 ft | -731.30 | ft-k |
| Moment | 240 ft | 411.30 | ft-k |
| Shear | 150 ft | 24.38 | k |
| Shear | 150 ft | -24.38 | k |
| *DW* | | | |
| Moment | 60 ft | 395.40 | ft-k |
| Moment | 150 ft | -703.10 | ft-k |
| Moment | 240 ft | 395.40 | ft-k |
| Shear | 150 ft | 23.44 | k |
| Shear | 150 ft | -23.44 | k |

Table 10: Unfactored Live Load Shear and Moment Effects

|  |  |  |  |
| --- | --- | --- | --- |
| Shear and Moment Effects from Visual Analysis | | | |
| *HL-93 Design Truck (Short)* | | | |
| Load Type | Location | Magnitude | Units |
| Moment | 60 ft | 1962.00 | ft-k |
| Moment | 150 ft | -1021.00 | ft-k |
| Moment | 240 ft | 1962.00 | ft-k |
| Shear | 150 ft | 69.16 | k |
| Shear | 150 ft | -69.46 | k |
| *HL-93 Design Truck (Long)* | | | |
| Moment | 60 ft | 1743.00 | ft-k |
| Moment | 150 ft | -980.30 | ft-k |
| Moment | 240 ft | 1743.00 | ft-k |
| Shear | 150 ft | 64.26 | k |
| Shear | 150 ft | -64.26 | k |
| *HL-93 Dual Truck* | | | |
| Moment | 60 ft | 3367.00 | ft-k |
| Moment | 150 ft | -3798.00 | ft-k |
| Moment | 240 ft | 3367.00 | ft-k |
| Shear | 150 ft | 154.30 | k |
| Shear | 150 ft | -154.30 | k |
| *HL-93 Design Tandem* | | | |
| Moment | 60 ft | 2884.00 | ft-k |
| Moment | 150 ft | -2519.00 | ft-k |
| Moment | 240 ft | 2884.00 | ft-k |
| Shear | 150 ft | 109.50 | k |
| Shear | 150 ft | -109.50 | k |
| *Fatigue (HL-93)* | | | |
| Moment | 60 ft | 1933.00 | ft-k |
| Moment | 150 ft | -1083.00 | ft-k |
| Moment | 240 ft | 1933.00 | ft-k |
| Shear | 150 ft | 70.33 | k |
| Shear | 150 ft | -70.33 | k |
| *Construction Live Load* | | | |
| Moment | 60 ft | 170.80 | ft-k |
| Moment | 150 ft | -303.80 | ft-k |
| Moment | 240 ft | 170.80 | ft-k |
| Shear | 150 ft | 10.13 | k |
| Shear | 150 ft | -10.13 | k |

* The maximum live load moment and shear effects, including dynamic load allowance and impact factor as applicable, are shown. *IES Visual Analysis* applied the HL-93 Design Load to the model (Design Truck, Design Tandem, Dual Truck, and Design Lane) such that the maximum effects were produced.

Live Load Distribution Factors

* Live loads are distributed to individual girders according to the approximate methods specified in AASHTO 4.6.2.2. The number of design lanes, NL, was assumed to equal two.

Table 11: Distribution Factors

|  |  |  |
| --- | --- | --- |
| Distribution Factors | | |
| DF, 2 Lanes (Strength/Service) | 0.688 | Lanes |
| DF, 1 Lane (Fatigue) | 0.688 | Lanes |
| LL+IM (Strength) | 1.33 | |
| IM (Fatigue) | 1.15 | |

Effective Width of Concrete Deck

* As specified in Article 6.10.1.1.1e, the effective flange width is to be determined as specified in Article 4.6.2.6. The individual webs of the tub girder must be initially considered separately since one web is an exterior web and the other is an interior web. According to Article 4.6.2.6, for an exterior web, the effective flange width may be taken as one-half the effective width of the adjacent interior girder, plus the full width of the overhang.

Table 12: Effective Width of Concrete Deck

|  |  |  |
| --- | --- | --- |
| Effective Width of Concrete Deck | | |
| beff. int. web | 61.98 | in. |
| beff. ext. web | 72.00 | in. |
| beff. girder | 133.98 | in. |

Investigate Service Limit State

* The factored service force effects were not directly calculated, but were investigated in the design of the post-tensioning, deck, and overhang.

Investigate Strength Limit State

* The factored moments used to check the Strength I Limit State are displayed in Table 13.

Table 13: Factored Moments - Strength Limit State

|  |  |  |
| --- | --- | --- |
| Factored Moments - Strength I | | |
| M+ (0.4 Span 1) | 14941.20 | ft-k |
| M- (Pier) | -23064.91 | ft-k |
| M+ (0.6 Span 2) | 14941.20 | ft-k |

Check Flexure and Constructability – Positive Bending

* The area of the concrete haunch was neglected in the calculation of girder section properties, but the height of the concrete haunch was considered.
* The upper limit of unbraced length outlined in AASHTO 6.10.1.6-2 was determined to be 54 feet. An unbraced length of 0.25L=37.5 feet was chosen based on engineering judgment. Conventional mild deck reinforcement, diaphragms, and stiffeners were not considered in this design.
* The plastic moment capacity of the girder was determined using AASHTO Article D6.1. The plastic neutral axis was calculated to be in the top flange. Thus, Case II in Table D6.1-1 was used to determine the plastic moment capacity and location of the plastic neutral axis.

Table 14: Component Forces - Positive Bending

|  |  |
| --- | --- |
| Component Forces & Details | |
| Top Flange, Pc (kips) | 4800.000 |
| Web, Pw (kips) | 3000.000 |
| Bottom Flange, Pt (kips) | 4800.000 |
| Slab, Ps (kips) | 4099.788 |

Table 15: Location of Component Forces - Positive Bending

|  |  |
| --- | --- |
| Location of Component Forces | |
| Ps (from PNA) | 7.771 |
| Pt (from PNA) | 61.229 |
| Pw (from PNA) | 30.229 |
| Pc (from PNA) | 0.771 |

Table 16: Plastic Moment Capacity - Positive Bending

|  |  |
| --- | --- |
| Plastic Moment Capacity | |
| 420272.31 | k-in |
| 35022.69 | k-ft |

* In composite bending, it was necessary to determine MAD, which is the additional moment that must be applied to the short-term composite section to cause nominal yielding in either steel flange, as per AASHTO Eq. D6.2.2-1:
* The yield moment of the composite section in positive bending was determined using AASHTO Eq. D6.2.2-2:
* Similarly, MAD and the yield moment of the composite section in negative bending were determined using AASHTO Eq. D6.2.2-1 and D6.2.2-2 with SLT and SST representing the steel section only, neglecting the strength contribution of the concrete slab. These calculations were facilitated using the *Data > What If > Goal Seek* function in *Microsoft Excel*. The factored MD1, MD2, and MLL+IM and *Goal Seek* results are tabulated below.

Table 17: Factored Loadings

|  |  |  |  |
| --- | --- | --- | --- |
| Loadings | MD1 (in-k) | Positive Bending | 101355.000 |
|  | Negative Bending | -180225.000 |
| MD2 (in-k) | Positive Bending | 13286.700 |
|  | Negative Bending | -23625.300 |
| MLL+IM (in-k) | Positive Bending | 7836.693 |
|  | Negative Bending | -8839.845 |
| Yield Strength (ksi) for Goal Seek | | 50 |
| MAD (Positive Bending) | Calc. M\_AD (k-in)  (Goal Seek) | **204442.848** |
| M\_AD (k-ft) | **17036.904** |
| Yield Strength (ksi) for Goal Seek | | 50 |
| MAD (Negative Bending) | Calc. M\_AD (k-in)  (Goal Seek) | **-84549.700** |
| M\_AD (k-ft) | **-7045.808** |

* The yield moment of the composite section in positive bending is shown below. Also, M­yt in negative bending was taken about the tension flange since mild steel reinforcement was not considered.

Table 18: Yield Moments

|  |  |  |
| --- | --- | --- |
| Yield Moment (0.4L) - Positive Bending | | |
| My | 26,590.37 | ft-k |
| Yield Moment (1.0L) - Negative Bending | | |
| My | -24,033.33 | ft-k |

Girder Constructability Check – Positive Moment Region

* Article 6.11.3 directs the engineer to Article 6.10.3 for the constructability checks of tub girders. For critical stages of construction, the provisions of Articles 6.10.3.2.1 through 6.10.3.2.3 shall be applied to the top flanges of the box girder. The non-composite bottom box flange in compression or tension shall satisfy requirements specified in Article 6.11.3.2
* Calculate the maximum flexural stresses in the flanges of the steel section due to the factored loads resulting from the application of steel self-weight and the assumed full deck-placement (DC1). As specified in Article 6.10.1.6, for design checks where the flexural resistance is based on lateral torsional buckling, fbu is to be determined as the largest value of the compressive stress throughout the unbraced length in the flange under consideration, calculated without consideration of flange lateral bending. For design checks where the flexural resistance is based on yielding, flange local buckling or web bend buckling, fbu may be determined as the stress at the section under consideration. The brace points were assumed to be located at intervals of 70 ft., as previously stated.
* In addition to the applied steel, permanent metal deck forms, and concrete self-weight loads, it is pertinent to assume a construction live loading (CLL) on the structure during placement of the concrete deck, as discussed in the load calculations section. In the STRENGTH I load combination; a load factor of 1.5 is applied to all construction loads, in accordance with Article 3.4.2.

Table 19: Check Flange Stress - Positive Bending

|  |  |  |
| --- | --- | --- |
| Check Flange Stresses - Positive Bending | | |
| fbu\_DC\_1 (top flange) | -17.572 | ksi |
| fbu\_DC\_1 (bottom flange) | 17.572 | ksi |
| fbu\_CLL (top flange) | -0.533 | ksi |
| fbu\_CLL (bottom flange) | 0.533 | ksi |
| fbu (top flange) | -18.105 | ksi |
| fbu (bottom flange) | 18.105 | ksi |

* The change in the horizontal component of the web shear in the web along the span acts as a lateral force in the flanges of the box girder. Under initial non-composite dead load DC1, the lateral force due to shear is assumed to be distributed to the top flanges of the closed box girder. To simplify the calculations for this example, it will conservatively be assumed that the entire DC1 horizontal component of web shear is applied to the top flanges. The change in vertical shear force, equal to the lateral load on the top flanges, is constant and is equal to the change in DC1 shear force in the girder measured at adjacent supports divided by the span length.

Table 20: Top Flange Lateral Bending due to Web Shear

|  |  |  |
| --- | --- | --- |
| Top Flange Lateral Bending due to Horizontal Component of Web Shear | | |
| Change in DC1 Girder Shear over Span | 5.340 | k/ft |
| Horizontal Component of Web Shear per Top Flange | 2.670 | k/ft |
| MLAT (assuming bracing at 70 ft.) | 50.063 | k-ft |
| Section Modulus of Top Flange | 768.000 | in.^3 |
| fLAT | 0.978 | ksi |

* The top flange lateral bending due to overhang loads was also considered. Although the brackets are typically spaced at 3 to 4 feet along the exterior girder, all bracket loads except for the finishing machine load are assumed to be applied uniformly. For this example, the bracket is assumed to extend near the edge of the deck overhang. Therefore it is assumed that half the deck overhang weight is placed on the exterior girder web and half the weight is placed on the overhang brackets. Conservatively, one-half the deck haunch weight was included in the total overhang weight.
* Construction loads, or dead loads and temporary loads that act on the overhang only during construction, were assumed as follows.

Table 21: Top Flange Lateral Bending due to Deck Overhang Loads

|  |  |  |
| --- | --- | --- |
| Top Flange Lateral Bending due to Deck Overhang Loads | | |
| Deck Overhang Weight | 281.250 | lb/ft |
| Overhang Deck Forms | 40.000 | lb/ft |
| Screed Rail | 85.000 | lb/ft |
| Railing | 25.000 | lb/ft |
| Walkway | 125.000 | lb/ft |
| Finishing Machine | 3000.000 | lb |
| FLAT/P | 0.800 | |
| *Strength I - Dead Loads* | | |
| p | 764.063 | lb/ft |
| FLAT | 611.250 | lb/ft |
| MLAT | 11.461 | k-ft |
| *Finishing Machine* | | |
| P | 4500.000 | lb |
| FLAT | 3600.000 | lb |
| MLAT | 6.750 | ft-k |
| fLAT (top flange) | 0.105 | ksi |
| Deck Overhang Total | 1.083 | ksi |

* Top flange lateral bending amplification was also considered in this design. As specified in Article 6.10.1.6, for design checks where the flexural resistance is based on lateral torsional buckling, the stress, fl, is to be determined as the largest value of the stress due to lateral bending throughout the unbraced length in the flange under consideration. For design checks where the flexural resistance is based on yielding or flange local buckling, fl may be determined as the stress at the section under consideration. For simplicity in this example, the largest value of fl within the unbraced length will conservatively be used in all design checks. fl was taken as positive in sign in all resistance equations. AASHTO Article 6.10.1.6 was used to determine the stress.

Table 22: Top Flange Lateral Bending Amplification

|  |  |  |
| --- | --- | --- |
| Top Flange Lateral Bending Amplification | | |
| Depth of Web in Compression, Dc | 30.000 | in. |
| rt | 13.509 | in. |
| Lp | 27.112 | ft. |
| Cb (conservative) | 1.0 | |
| Lb (Assumed) | 37.500 | ft. |
| Lb (Limit) 6.10.1.6-2 | 54.066 | ft. |
| Fcr | 257.943 | ksi |
| Amplification Factor, AF | 1.000 | OK |
| fLAT, including AF | 2.061 | ksi |
| Stress Limit (6.10.1.6-1) | 30.000 | ksi |

* Article 6.11.3.2 directs the engineer to the provisions of Article 6.10.3.2 for top flange constructability checks. Article 6.10.3.2.1 requires that discretely braced flanges in compression satisfy the following requirements, except that for slender-web sections, Eq. (6.10.3.2.1-1) need not be checked when flis equal to zero.
* Article 6.11.3.2 requires that the non-composite tub flange (bottom flange) in tension satisfy:

Table 23: Local Buckling Resistance Check - Positive Bending

|  |  |  |
| --- | --- | --- |
| 2Dc/tw | 120.00 | NOT SLENDER |
| 5.7√(E/Fyc) | 137.274 | |
| *Top Flange Local Buckling Resistance* | | |
| λf | 12.000 | |
| λpf | 9.152 | NONCOMPACT |
| λrf | 13.487 | |
| Fyr | 35.000 | ksi |
| Fnc | 40.144 | ksi |

* Lateral torsional buckling resistance was also considered, as per AASHTO Article 6.10.8.2.3.

Table 24: Lateral Torsional Buckling Resistance - Positive Bending

|  |  |  |
| --- | --- | --- |
| *Top Flange Lateral Torsional Buckling Resistance (6.10.8.2.3)* | | |
| Lr | 101.803 | ft. |
| Fnc (6.10.8.2.3) | 47.914 | ksi |
| Rh\*Rb\*Fyc | 50.000 | ksi |
| Fnc | 47.914 | ksi |
| Fnc (governing) | 40.144 | ksi |

* Web bend-buckling resistance was also considered, as per AASHTO Article 6.10.1.9.1.

Table 25: Web Bend-Buckling Resistance - Positive Bending

|  |  |  |
| --- | --- | --- |
| *Web Bend Buckling Resistance (6.10.1.9)* | | |
| k | 36.000 | |
| Fcrw | 65.250 | ksi |
| F­crw Limit | 50.000 | ksi |
| Fcrw USE | 50.000 | ksi |

* Since all the criteria were determined, the top flange constructability checks were made according to AASHTO Article 6.10.3.2.1:

Table 26: Top Flange Constructability Check - Positive Bending

|  |  |  |  |
| --- | --- | --- | --- |
| Constructability Checks - Positive Bending | | |  |
| *Top Flange Constructability Checks (6.10.3.2.1)* | | |  |
| f\_bu+f\_lat (6.10.3.2.1-1) | 19.188 | ksi | OK |
| Ratio | 0.3838 | |  |
| f\_bu+(1/3)f\_lat (6.10.3.2.1-2) | 18.466 | ksi | OK |
| Ratio | 0.4600 | |  |
| f\_bu < F\_crw | 18.105 | ksi | OK |
| Ratio | 0.3621 | |  |

* Non-composite box flanges (bottom flanges) in tension, must satisfy AASHTO Eq. 6.11.3.2-3:
* The term fv is the St. Venant torsional shear stress in the flange due to factored loads at the section under consideration. However, in accordance with Article C6.11.2.3, if the provisions of Article 6.11.2.3 are satisfied, shear due to St. Venant torsion and secondary distortional bending stress effects may be neglected if the width of the box flange does not exceed one-fifth the effective span defined in Article 6.11.1.1. For continuous spans, the effective span length is to be taken as the distance between points of permanent load contraflexure, or between a simple support and a point of permanent load contraflexure, as applicable. Since the above conditions were satisfied, fv=0 in this case.

Table 27: Bottom Flange Constructability Check - Positive Bending

|  |  |  |  |
| --- | --- | --- | --- |
| *Bottom Flange Constructability Checks (6.11.3.2)* | | |  |
| fv | 0.0 | ksi |  |
| Δ | 1.000 |  |  |
| fbu | 18.105 | ksi |  |
| Rh\*Fyf\*Δ | 50.000 | ksi | OK |

* The service limit state was then checked for positive moment. Article 6.11.4 directs the Engineer to Article 6.10.4, which contains provisions related to the control of elastic and permanent deformations at the Service Limit State. For the sake of brevity, only the calculations pertaining to permanent deformations will be presented for this design.
* Article 6.10.4.2 contains criteria intended to control permanent deformations that would impair rideability. As specified in Article 6.10.4.2.1, these checks are to be made under the SERVICE II load combination.
* Article 6.10.4.2.2 requires that, flanges of composite sections must satisfy AASHTO Equations 6.10.4.2.2-1 and 6.10.4.2.2-2.

Top Flange:

Bottom Flange:

* The term ff is the flange stress at the section under consideration due to the SERVICE II loads calculated without consideration of flange lateral bending. The fl­ term, the flange lateral bending stress, in Eq. (6.10.4.2.2-2) shall be taken equal to zero, in accordance with Article 6.11.4. A resistance factor is not included in these equations because Article 1.3.2.1 specifies that the resistance factor be taken equal to 1.0 at the service limit state.
* With the exception of composite sections in positive flexure in which the web satisfies the requirement of Articles 6.11.2.1.2 and 6.10.2.1.1 (D/tw ≤150), web bend-buckling of all sections under the SERVICE II load combination is to be checked as follows:
* The term fc is the compression-flange stress at the section under consideration due to the SERVICE II loads calculated without consideration of flange lateral bending, and Fcrw is the nominal elastic bend-buckling resistance for webs determined as specified in Article 6.10.1.9. Because the girder under consideration is a composite section subject to positive flexure satisfying Article 6.11.2.1.2, Eq. (6.10.4.2.2-4) need not be checked. An explanation as to why these particular sections are exempt from the above web bend-buckling check is given in Article C6.10.1.9.1.

Table 28: Service Limit State Check - Positive Bending

|  |  |  |  |
| --- | --- | --- | --- |
| Service Limit State Check - Positive Bending | | |  |
| *Top Flange* | | |  |
| 0.95\*R\_h\*F\_yf | 47.5 | ksi |  |
| f\_f | 17.283 | ksi | OK |
| Ratio | 0.3639 | |  |
| *Bottom Flange* | | | |
| f\_l | 0 | ksi |  |
| f\_f+(1/2)\*f\_l | 21.053 | ksi | OK |
| Ratio | 0.4432 | |  |

* It should be noted that in accordance with Article 6.11.4 redistribution of negative moment due to the Service II loads at the interior-pier sections in continuous span flexural members using the procedures specified in Appendix B shall not apply to box girder sections. The applicability of the Appendix B provisions to box girder sections has not been demonstrated; hence the procedures are not permitted for the design of box girder sections.
* Determine if the girder under consideration qualifies as a compact section. According to Article 6.11.6.2.2, composite sections in positive flexure qualify as compact when:

1. The specified minimum yield strengths of the flanges and web do not exceed 70 ksi
2. The web satisfies the requirement of Article 6.11.2.1.2 such that longitudinal stiffeners are not required (i.e. D/tw  \_150)
3. The section is part of a bridge that satisfies the requirements of Article 6.11.2.3 (Special Restrictions for use of live load distribution factors)
4. The tub flange (bottom flange) is fully effective as specified in Article 6.11.1.1 (i.e. bottom flange bf less than one-fifth effective span)
5. The section satisfies the following web-slenderness limit:

Table 30: Compactness Check - Positive Bending

|  |  |
| --- | --- |
| Check Compactness in Composite Condition (Output is 1 for Yes, 0 for No) | |
| Is the yield stress of the compression flange less than 70ksi? | 1 |
| Is the yield stress of the tension flange less than 70ksi? | 1 |
| Is 2Dcp/tw less than or equal to 3.76√E/Fy? | 1 |
| Is D/tw less than or equal to 150? | 1 |
| Is the girder straight (not curved)? | 1 |
| Does section meet all compactness criteria? | Yes |

* At the strength limit state, compact composite sections in positive flexure must satisfy the provisions of Article 6.11.7.1.
* The nominal flexural resistance of the section shall be taken as specified in Article 6.10.7.1.2, except that for continuous spans, the nominal flexural resistance shall always be subject to the limitation of Eq. (6.10.7.1.2-3). According to the provisions of Article 6.10.7.1.2, the nominal flexural resistance of compact composite sections in positive flexure is determined as follows:
* In continuous spans, the nominal flexural resistance of the section is also limited to the following:
* Thus:

Table 30: Plastic Dimensions - Positive Bending

|  |  |
| --- | --- |
| Plastic Dimensions (Composite) | |
| Distance from Top of Deck to PNA, Dp (in.) | 12.771 |
| Total Depth of Composite Section, Dt (in.) | 76.500 |
| Depth of Web in Comp. at Plastic Moment, Dcp (in.) | 0.000 |
| Depth of Web in Comp. in Elastic Range, Dc (in.) | 30.000 |

Table 31: Nominal Moment Capacity - Compact Sections (Positive Bending)

|  |  |
| --- | --- |
| Nominal Moment Capacity - Compact Sections (6.10.7.1) | |
| Ratio of Dp/Dt | 0.167 |
| Mn if Dp/Dt <= 0.1 | N/A |
| Mn if Dp/Dt> 0.1 | 31243.33 |

Table 32: Flexural Capacity - Positive Bending

|  |  |  |  |
| --- | --- | --- | --- |
| Check Flexural Capacity - Positive Bending | | |  |
| Mn | 31243.33 | ft-k |  |
| Mu | 14941.20 | ft-k | OK |
| Ratio | 0.4782 | | |

Check Flexure and Constructability – Negative Bending

* The bottom flange, in regions of negative flexure, shall satisfy the requirements of Eqs. 6.11.3.2-1 and 6.11.3.2-2 for critical stages of construction. Generally these provisions will not control because the size of the bottom flange in negative flexure regions is normally governed by the Strength Limit State. In regard to construction loads, the maximum negative moment reached during the deck-placement analysis, plus the moment due to the self-weight, typically do not differ significantly from the calculated DC1 negative moments assuming a single stage deck pour.
* The deck pour sequence and the application of wind loads are not considered in this example. It is assumed for this design that the application of the concrete deck occurs all at once for the purpose of the constructability checks.

Table 33: Check Flange Stress - Negative Bending

|  |  |  |
| --- | --- | --- |
| Check Flange Stresses - Negative Bending | | |
| fbu\_DC\_1 (top flange) | 31.246 | ksi |
| fbu\_DC\_1 (bottom flange) | -31.246 | ksi |
| fbu\_CLL (top flange) | 0.948 | ksi |
| fbu\_CLL (bottom flange) | -0.948 | ksi |
| fbu (top flange) | 32.194 | ksi |
| fbu (bottom flange) | -32.194 | ksi |

* For the flexural resistance of unstiffened flanges in compression, calculate the nominal flexural resistance of the bottom flange in compression, Fnc, in accordance with Article 6.11.8.2. In computing Fnc for constructability, the web load-shedding factor, Rb, shall be taken as 1.0.

Table 34: Constructability Check - Negative Bending

|  |  |  |  |
| --- | --- | --- | --- |
| Constructability Checks - Negative Bending | | |  |
| fbu + flat (6.10.3.2.2-1) | 33.277 | ksi | OK |
| Ratio | 0.6655 | |  |
| *Bottom Flange - Unstiffened Flange (6.11.8.2.2)* | | |  |
| λf | 12.000 |  |  |
| k | 4.000 |  |  |
| ks | 5.340 |  |  |
| fv | 0.000 |  |  |
| Δ | 1.000 |  |  |
| λp | 27.454 |  |  |
| Fyr | 35.000 | ksi |  |
| λr | 54.691 |  |  |
| Fcb | 50.000 | ksi |  |
| 1.12\*√((E\*ks)/Fyc) | 62.331 |  |  |
| 1.40\*√((E\*ks)/Fyc) | 77.913 |  |  |
| Fcv | 29.000 | ksi |  |
| Fnc | 50.000 | ksi | OK |
| Ratio | 0.644 |  |  |

* The web bend-buckling resistance shall be compared with the maximum compressive stress in the bottom flange. Determine the nominal elastic web bend-buckling resistance over the girder over the pier according to the provisions of Article 6.10.1.9.1.

Table 35: Web Bend-Buckling Resistance - Negative Bending

|  |  |  |  |
| --- | --- | --- | --- |
| Web Bend-Buckling Resistance - Negative Bending (6.10.1.9.1) | | |  |
| Dc | 30.000 | in |  |
| k | 36.000 |  |  |
| Fcr | 65.250 | ksi |  |
| Fcr USE | 50.000 | ksi | OK |
| fbu | -32.194 | ksi |  |

* In checking the strength limit state of composite sections in negative flexure, Article 6.11.6.2.3 directs the Engineer to Article 6.11.8. Furthermore, Article 6.11.6.2.3 states the provisions of Appendix A shall not apply, nor is redistribution of negative moment per Appendix B.
* At the strength limit state, box flanges (bottom flanges) in compression shall satisfy:
* At the Strength Limit State, the top flanges in tension continuously braced by the deck, shall satisfy:
* Determine the nominal flexural resistance of the bottom flange in compression, Fnc, in accordance with Article 6.11.8.2.
* The results of the aforementioned process are shown below:

Table 36: Strength Limit State Check - Negative Bending

|  |  |  |  |
| --- | --- | --- | --- |
| Strength Limit State - Negative Bending | | |  |
| fbu (top flange) | 44.803 | ksi |  |
| fbu (bot. flange) | -48.171 | ksi |  |
| Dc | 31.578 | in |  |
| 2Dc/tw | 126.312 |  |  |
| λrw | 137.274 |  |  |
| Rb | 1.000 |  |  |
| awc | 0.3289 |  |  |
| Fnc | 50.000 | ksi | OK |
| Fnt | 50.000 | ksi | OK |

* Thus, the negative bending region criteria were satisfied.

1. Master’s student at the University of Delaware [↑](#footnote-ref-1)