COMPARATIVE ANALYSIS OF PLATE GIRDER DESIGNS ON NON-COMPOSITE BRIDGES BETWEEN AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS 2017 CODE WITH SNI 1729:2015 CODE

Donald, ESSEN,
Universitas Mercu Buana Jakarta, Indonesia,
donaldessenstmt@gmail.com

Nurul Musyafa Ulul, HIDAYAH,
Universitas Mercu Buana Jakarta, Indonesia,
syafagihan@gmail.com

ABSTRACT

This study aims to the structural design of non-composite plate girders using AASHTO LRFD Bridge Design Specifications 2017 code compared to SNI 1729:2015 code. The span of the bridge used as the object of study is 40 meters with a width of 10 meters. In this study, plate girders are designed based on AASHTO code and SNI code, then also given the loading according to SNI 1725:2016 code, and in the analysis of the structure using CSi Bridge software to get the value of internal forces i.e. Moment Force \( (M_u) \) of 3595.38 kNm and Shear Force \( (V_u) \) of 449.9968 kNm. The results obtained from this study are the non-composite bridge plate girder designed with AASHTO LRFD Bridge Design Specifications 2017 and SNI 1729:2015 obtained the stability requirements of strong boundary conditions flexure design. Then obtained Nominal Moment value \( (\Omega M_n) \) of 8016.843 kNm for AASHTO LRFD Bridge Design Specifications 2017 and Nominal Moment value \( (\Omega M_n) \) of 6081.97 kNm for SNI 1729:2015. From the values obtained it can be concluded that the two regulations produce a safe and strong plan as per the applicable provisions namely Moment \( (M_u < \Omega M_n) \).

Keywords: girder, AASHTO LRFD Bridge Design Specifications 2017, SNI 1729:2015, non-composite, flexure design

INTRODUCTION

Indonesia is an archipelago with an area of water that reaches 64.97% of the total area (Bakosurtanal, 2014) (1). But that is not an obstacle for the central government to carry out the mandate of the nation listed in the Pancasila precisely in the 5th precept that reads "Social Justice for All Indonesian People".

Engineers in the construction world are expected to be able to actively participate in this national development effort. One type of construction that will be used in these efforts is a bridge. The bridge consists of many models and of course related to their respective functions. From a small bridge that serves to cross people located on the highway to a large bridge connecting inter-island that can be passed by motorized vehicles, such as motorcycles, cars, trucks, buses and so forth.

In its planning, the bridge itself is divided into several construction components such as:

a) Abutment
b) Bearings
c) Pier
d) Pile cap
e) Bored Pile
f) Girder (Composite / Non-Composite)
g) Deck
h) Highway
i) Sidewalks
j) Tension
Of all the construction components, of course there are planning rules that are binding, standard and systematic. One of them is the component construction of the plate girders.

Plate girder is a large beam that is made from the arrangement of plate elements that are joined with a connecting tool to get a more efficient arrangement of material than that obtained with a wrought beam (rolled beam). Steel material has mechanical properties that are strong against compressive / tensile forces, but it should be noted that the stability of bending. Basically, construction of large bridges will require relatively long stretches and it is also likely to be without a column / pier in the middle, therefore this plate girder selection is one of the solutions in implementing bridge construction work.

In the design of this steel frame bridge, it will use two bridge structure design regulations, namely using the AASHTO LRFD Bridge Design Specifications (American) and SNI (Indonesia), which in the end can be seen differences and similarities in the results of the design. The two construction regulations together show how to calculate and analyze plate girders. The differences and design similarities of the two reference rules will be discussed in my study this time.

The bridge is a part of the road that functions to connect between two separate roads due to obstacles such as rivers, valleys, seas, highways, and railroad tracks. The bridge is very vital function of human life, and has an important meaning for everyone. However, the level of importance is not the same for everyone, so it will be an interesting study material (Bambang Supriyadi, 2007) (2).

RESEARCH METHODOLOGY

The method that the author uses in this research is carried out by the method of literature study. Literature study is a description of the theoretical foundations relating to non-composite bridge planning. Literature studies are sourced from books, regulatory standards, and journals related to structural planning for non-composite bridges.
Fig. 2 Bridge Cross Section

Assumption Technical Data
a. Bridge type: Non-composite
b. Total bridge width: 10 m
c. Span length: 40 m

RESULTS AND DISCUSSIONS

Calculation of Loading Non-Composite Steel
1. Self Weight (MS)
   1.1 I Steel Girder Weight
   The weight of the steel girder I was calculated automatically by the program, with \(\gamma_{\text{steel}} = 78.50 \text{ kN/m}^3\)
   1.2 Deck Weight
Deck has dimensions:
cross-sectional area \( A_{\text{deck}} = 0.4 \text{ m}^2 \)
Concrete specific gravity \( Y_{\text{concrete}} = 25 \text{ kN/m}^3 \)
Then the deck load \( = A_{\text{deck}} \times Y_{\text{beton}} = 10 \text{ kN/m} \)

1.3 Diaphragm Weight

Diaphragm has dimensions:
Diaphragm volume \( V = 0.006096 \text{ m}^3 \)
Steel specific gravity \( Y_{\text{steel}} = 78.5 \text{ kN/m}^3 \)

2. Additional Dead Load / Utilities (MA)

2.1 Load Pavement

Concrete specific gravity \( Y_{\text{concrete}} = 25 \text{ kN/m}^3 \)
wide sidewalk \( A_{\text{sidewalk}} = 0.189 \text{ m}^2 \)
Load of sidewalk \( q_{\text{sidewalk}} = 25 \times 0.198 = 4.73 \text{ kN/m (per side)} \)

2.2 Railing Loads

Concrete specific gravity \( Y_{\text{concrete}} = 25 \text{ kN/m}^3 \)
Wide railing concrete \( A_{\text{rall}} = 0.196 \text{ m}^2 \)
Load concrete railing \( q_{\text{rall}} = 25 \times 0.196 = 4.90 \text{ kN/m (per side)} \)

2.3 Asphalt Loads

Asphalt density \( Y_{\text{asphalt}} = 22.4 \text{ kN/m}^3 \)
Asphalt thickness \( t_{\text{asphalt}} = 0.05 \text{ m} \)
Road width \( b_{\text{road}} = 7 \text{ m} \)
Asphalt load \( q_{\text{asphalt}} = 22.4 \times 0.05 \times 7 = 7.84 \text{ kN/m (per entire bridge width)} \)

2.4 Concrete Screed Loads

Concrete specific gravity \( Y_{\text{concrete}} = 25 \text{ kN/m}^3 \)
Concrete screed area \( A_{\text{screed}} = 0.35 \text{ m}^2 \)
Concrete screed load \( q_{\text{screed}} = 25 \times 0.35 = 8.75 \text{ kN/m (per entire bridge width)} \)
Then the total MA load = 35.85 kN/m (per entire bridge width)
And for the total MA load per girder = \( \frac{35.85}{4} = 8.96 \text{ kN/m} \)

3. "T" Truck Load Calculation (TT)

The truck load 'T' used is in accordance with SNI 1725: 2016[8] as shown

Fig. 4 Truck Load 'T'

Dynamic load factor (FBD) for BGT is taken FBD = 0.300
Front wheel weight \( P_{\text{frd}} = 25,000 \text{ kN} \)
Rear wheel weight \( P_{\text{frb}} = 112,500 \text{ kN} \)
Weight of front wheel + FBD \( P_1 = 32.5 \text{ kN} \)
Rear wheel weight + FBD \( P_2 = 146.25 \text{ kN} \)

4. Calculation of Lane Load 'D'

The vehicle live load consists of lane load with the component load evenly distributed BTR and BGT line load as shown in Figure 4.3 BGT taken at 49 kN/m. The lateral distribution of the "D" lane load is placed with an intensity of 100%. The q value of BTR is determined from the following equation:
If \( L \leq 30 \text{ m} \) \( \Rightarrow q = 9.0 \text{ kPa} \)
If \( L > 30 \text{ m} \) \( \Rightarrow q = 9.0 \left( 0.5 + \frac{15}{L} \right) \text{ kPa} \)
Lane density factors must be calculated according to Table 3.1

**Table 3.1 Lane Density Factors**

| Jumlah lajur yang  | faktor kepadatan |
|--------------------|------------------|
| dibebani           | lajur            |
| 1                  | 1                |
| ≥ 2                | 1                |

Evenly distributed load intensity (BTR) \(q = 7,875 \text{ KPa} = \text{kN/m}^2\)

Centralized line load intensity (BGT) \(p = 49,000\)

Dynamic load factor (FBD) for BGT is taken \(FBD = 0.300\)

Load evenly distributed (BTR) \(q = 14.40 \text{ kN/m}\)

Central line load (BGT) + (FBD) \(p = 109.76 \text{ kN/m}\)

5. **Pedestrian Loads**
   All sidewalk components wider than 600 mm must be planned to carry pedestrian loads with an intensity of 5 kPa.
   a. Sidewalk width \(b_{sw} = 1.50 \text{ m}\)
   b. Intensity of pedestrian loads \(PTP = 5.00 \text{ kN/m}^2\)

**Application Load On Steel Girder Composite Modeling**

1. Application load of asphalt on the bridge
   **Fig 6. Modeling loading asphalt 5 cm**

2. Application load on the left side of the pedestrian bridge
   **Fig. 7 Modeling 3D pedestrian loading Left**
3. Application load on the right side of the pedestrian bridge

![Fig. 8 Modeling pedestrian loading right]

4. Applications load concrete pavement on bridges

![Fig. 9 Modeling loading MA - pavement]

**Load Combination**

The combination of loading for Strong Boundary Conditions must comply with SNI 1725: 2016 as given in Table 3.2 as follows:

| Table 3.2 Combination of Load and Load Factor |
|---------------------------------------------|
| MA | EI | C0 | CNW | BI | EI |
|----|----|----|-----|----|----|
| 7  | 1  | 0  | 0   | 0  | 0  |
| 7  | 1  | 0  | 0   | 0  | 0  |
| 7  | 1  | 0  | 0   | 0  | 0  |
| 7  | 1  | 0  | 0   | 0  | 0  |

**Cross Section Proportion**

Determining the Cross-Section Proportion

\[
\frac{d}{L} = \frac{1}{25}
\]

\[
d = \frac{L}{30} = \frac{40000 \text{ mm}}{25} = 1600 \text{ mm}
\]

Raise the 10% height of the girder for SNI vehicle loads obtained:

\[
d + (dx10%) = 1600 + (1600\times10%) = 11760 \text{ mm}
\]

Then take grades \(d = 1800 \text{ mm}\)

\(t_s = 250 \text{ mm}\)

Take \(S = 2.5 \text{ m}\) & \(S' = 0.75\)

Flange width is controlled when handling & transportation

\(b_{fc} \geq \frac{L}{85} \text{ mm} \Rightarrow L = \text{ shipping length} = 12 \text{ m}\)
The minimum flange thickness for a straight bridge is 19 mm, so take:

- Thick flange upper = 20 mm
- Thick flange bottom = 20 mm

So for web height D = 1800-20-20 = 1760 mm

Minimum web thickness of 12 mm, take \( t_w = 14 \) mm

Check the flange proportion requirements:

\[
b_f \geq \frac{D}{6} = \frac{1760}{6} = 293.33 \text{ mm}
\]

Use \( b_f = 350 \) for the upper and lower flanges

\[
\frac{b_f}{2t_f} \leq 9.2 \Rightarrow \frac{350}{2 \times 20} = 8.75 < 9.2 \text{ (top flanges NSBA)}
\]

\[
\frac{b_f}{2t_f} \leq 9.2 \Rightarrow \frac{350}{2 \times 20} = 8.75 < 9.2 \text{ (bottom flanges NSBA)}
\]

Then from the data above we get the following cross-section proportions:

- **Steel IWF 1800.300.14.20**
  - \( d = 1800 \) mm
  - \( b_f = 300 \) mm
  - \( t_w = 14 \) mm
  - \( t_f = 20 \) mm
  - \( t_b = 20 \) mm
  - \( L = 40 \) m
  - \( F_{c'} = 40 \) MPa
  - \( F_y = 345 \) MPa
  - \( D_w = 1760 \) mm
  - \( E_c = 4700 \sqrt{f_{c'}} = 29725.41 \) MPa
  - \( E_s = 200000 \) MPa
  - \( n = \frac{E_s}{E_c} = 6.728 \)

**Examination of Proportion Limits for Profile Cross-section**

The cross-section proportion must be checked to ensure the stability of the profile meets the requirements. Determination of the cross-section proportion set in the AASHTO LRFD Bridge Design Specifications 2017 Article 6.10.2:

*In accordance with AASHTO LRFD Bridge Design Specifications 2017 article 6.10.2.1.1*

1. **Proportion of Body Plate Without Stiffener**
   
   Calculated as follows:
   
   \[
   \frac{D_w}{t_w} = \frac{1760}{14} = 125.71 \text{ mm}
   \]
   
   Check web stability = \( \frac{t_w}{b_w} \leq 150 \rightarrow 125.71 \leq 150 \), then including slim web.

2. **Proportion of Wing Plate Sections**
   
   In accordance with AASHTO LRFD Bridge Design Specifications 2017 article 6.10.2.2, for cross section of wing plate as follows:
   
   - **AASHTO LRFD 2017 6.10.2.2-1**
     
     \[
     \frac{b_f}{2t_f} \leq 12 \Rightarrow \frac{300}{6} = 50 \leq 12 \text{ ...OK}
     \]
   
   - **AASHTO LRFD 2017 6.10.2.2-2**
     
     \[
     b_f \geq \frac{D_w}{6} = \frac{1760}{6} = 293.33 \text{ mm} \rightarrow 0.3m \geq 0.293m \text{ ...OK}
     \]
   
   - **AASHTO LRFD 2017 6.10.2.2-3**
     
     \[
     t_f \geq 1.1t_w \rightarrow 1.1 \times 14 = 0.015m
     \]
     
     \[
     t_f = 0.020m
     \]
     
     \[
     0.020 \geq 0.015 \text{ ...OK}
     \]
   
   - **AASHTO LRFD 2017 6.10.2.2-4**
Modeling of Non-composite Bridges

Modeling of non-composite bridges was carried out with the help of CSI Bridge v20.2.0 program. Figure 10 shows the 3D appearance of the non-composite bridge model.

\[ \frac{I_{yc}}{I_{yt}} \leq 10 \]

\[ I_{yc} = \frac{t_f \cdot b_f^3}{12} = \frac{0.020 \cdot 0.0252}{12} = 0.000042 \text{ m}^4 \]

\[ I_{yt} = \frac{t_f \cdot b_f^3}{12} = \frac{0.020 \cdot 0.0252}{12} = 0.000042 \text{ m}^4 \]

\[ 0.1 \leq \frac{I_{yc}}{I_{yt}} \leq 10 \]

\[ 0.1 \leq 1 \leq 10 \]

Structure Analysis

The following results of structural analysis that work on non-composite bridges due to strong combinations resulting from modeling using CSiBridge software are as follows:

1. Moment Field in Girder Interior 1

The value of internal forces in the interior girder 1
\[ M_1 = -3595.38 \text{ kNm} \]

The value of internal forces in the interior girder 2
\[ M_2 = -3595.25 \text{ kNm} \]

The value of internal forces in the interior beam 1
\[ M_1 = -3593.4 \text{ kNm} \]

The value of internal forces in the interior beam 2
\[ M_2 = -3593.32 \text{ kNm} \]

The value of internal forces in the interior slab 1
\[ M_1 = -0.0118 \text{ kNm} \]

The value of internal forces in the interior slab 2
\[ M_2 = -0.0118 \text{ kNm} \]

From the above data, the highest Flexural Strength (\(M_u\)) value is taken, which is found in the interior of girder 1, which is: -3595.38 kNm
2. Slide Field On Girder Interior 1

Fig. 12 Shear Force Non-Composite Steel Structures

The value of the shear force on the interior of the girder 1
\[ V1 = -449.9968 \] (kNm)

The value of the shear force on the interior of the girder 2
\[ V2 = -449.9818 \] (kNm)

The value of the shear force on the interior beam 1
\[ V1 = -442.4905 \] (kNm)

The value of the shear force on the interior beam 2
\[ V2 = -442.4755 \] (kNm)

The value of the shear force on the interior of the slab 1
\[ V1 = -7.5063 \] (kNm)

The value of the shear force on the interior of the slab 2
\[ V2 = -7.5063 \] (kNm)

From the above data, the largest Shear Force \((V_u)\) value is taken, which is found in the interior of girder 1, which is: -449.9968 kNm

**Calculation of Inertia and Center of Cross Section**

In the calculation of inertia and cross-section emphasis is done using Microsoft Excel and obtained the following data:

**Non Composite Elastic Flexural Section Properties**

![Non Composite Elastic Flexural Section Properties](image)
**Non Composite Elastic Torsional Section Properties**

**Fig. 14 Non Composite Elastic Torsional Section Properties**

| Part | x | y | a | a/2 | Cross Section
|------|---|---|---|-----|----------------|
| Gross Area | ΣA = | 𝐴𝑡𝑓 + 𝐴𝑤 + 𝐴𝑏𝑓 = 36640.0 m m^2 |
| Gross moment of inertia about x | Σ (A x (y - Y𝑏)\(^2\) + I_0x | = 15866005333.3 mm^4 |
| Gross moment of inertia about y | Σ (A x (x - X)\(^2\) + I_0y | = 90402453.3 mm^4 |
| Girder top elastic section modulus about x | \( \frac{I_x}{(H+tcp-Yc.g.c)} \) | = 17628894.8 mm^3 |
| Girder bottom elastic section modulus about x | \( \frac{I_x}{(H+tcp-Yc.g.c)} \) | = 17628894.8 mm^3 |
| Girder left elastic section modulus about y | \( \frac{I_y}{(B ft/2)} \) | = 602683.0 mm^3 |
| Girder right elastic section modulus about y | \( \frac{I_y}{(B ft/2)} \) | = 602683.0 mm^3 |
| Girder plastic section modulus about x | \( \frac{M_{px}}{f_y \cdot 10^6} \) | = 22781600 m m^3 |
| Girder plastic section modulus about y | \( \frac{M_{py}}{f_y \cdot 10^6} \) | = 986240 m m^3 |
| Radius of gyration about x | \( \sqrt{\frac{\Sigma Ay}{\Sigma A}} \) | = 658 mm |
| Radius of gyration about y | \( \sqrt{\frac{\Sigma Ax}{\Sigma A}} \) | = 49.7 mm |
| Centroid from bottom | \( \frac{\Sigma Ay}{\Sigma A} \) | = 32976000 | 36640 | = 900 mm |
| Centroid from mid of web | \( \frac{\Sigma Ax}{\Sigma A} \) | = 0 | 36640 | = 0 mm |
| Torsional Constant | \( \sum \frac{(a.b.f)^3}{3} \) | = 3228106.7 mm |
| Warping Constant | \( \sum \frac{(a.b.f)^3}{3} \) | = 7128900000000 mm |

\[ \text{Girder moment of inertia about } x = \sum (A \times (y - Y_b)^2 + I_0x) = 9505200000.0 + 6360805333 = 15866005333.3 \text{ mm}^4 \]
\[ \text{Girder moment of inertia about } y = \sum (A \times (x - X)^2 + I_0y = 90402453.3 \text{ mm}^4 \]
\[ \text{Girder top elastic section modulus about } x = \frac{I_x}{(H+tcp-Yc.g.c)} = 17628894.8 \text{ mm}^3 \]
\[ \text{Girder bottom elastic section modulus about } x = \frac{I_x}{(H+tcp-Yc.g.c)} = 17628894.8 \text{ mm}^3 \]
\[ \text{Girder left elastic section modulus about } y = \frac{I_y}{(B ft/2)} = 602683.0 \text{ mm}^3 \]
\[ \text{Girder right elastic section modulus about } y = \frac{I_y}{(B ft/2)} = 602683.0 \text{ mm}^3 \]
\[ \text{Girder plastic section modulus about } x = \frac{M_{px}}{f_y \cdot 10^6} = 22781600 \text{ m m}^3 \]
\[ \text{Girder plastic section modulus about } y = \frac{M_{py}}{f_y \cdot 10^6} = 986240 \text{ m m}^3 \]
\[ \text{Radius of gyration about } x = \sqrt{\frac{\Sigma A y}{\Sigma A}} = 658 \text{ mm} \]
\[ \text{Radius of gyration about } y = \sqrt{\frac{\Sigma A x}{\Sigma A}} = 49.7 \text{ mm} \]
\[ \text{Centroid from bottom} = \frac{\Sigma A y}{\Sigma A} = \frac{32976000}{36640} = 900 \text{ mm} \]
\[ \text{Centroid from mid of web} = \frac{\Sigma A x}{\Sigma A} = \frac{0}{36640} = 0 \text{ mm} \]
\[ \text{Torsional Constant} = \sum \frac{(a.b.f)^3}{3} = \frac{3228106.7}{3} = 3228106.7 \text{ mm} \]
\[ \text{Warping Constant} = \sum \frac{(a.b.f)^3}{3} = \frac{71289000000000}{3} = 71289000000000 \text{ mm} \]
Section Classification
Straight bridges with non-composite cross sections must be checked for:

In Accordance with AASHTO LRFD Bridge Design Specification 2017 (6.10.6.2.2-2)

\[ f_{yf} = f_y = 345 \text{ MPa} \]

\[ f_{yf} \leq 485 \text{ MPa} \]

345 \leq 485 \text{ MPa} \ldots \text{OK}

In Accordance with AASHTO LRFD Bridge Design Specification 2017 (6.10.6.2.2-1)

\[ 2 \cdot \frac{D_{cp}}{t_w} \leq 3.76 \sqrt{\frac{E_s}{f_{c'} t'}} \]

Where:

\[ y_{stop} = 0.59 \text{ m} \]

\[ D_{cp} = y_{stop} - t_f = 0.57 \text{ m} \]

So,

\[ 2 \cdot \frac{D_{cp}}{t_w} = \frac{1140}{14} = 81.42 \text{ mm} \]

\[ 3.76 \sqrt{\frac{E_s}{f_{c'}}} = 265.87 \]

\[ 81.42 \leq 265.87 \ldots \text{OK} \]

Non-Composite Plate Girder Design Based on AASHTO LRFD Bridge Design Specification 2017

Check the State of the Cross Section

During construction

Due to flexure

1. Upper Flens
   Style that works:
   \[ M_{girder} = 3595.38 \text{ kN.m} \]
   \[ M_{plate} = 781.25 \text{ kN.m} \]
   \[ I_s = 15866005333.3 \text{ mm}^4 \]
   Tension on the upper wing:
   \[ \sigma_{topflange} = \frac{(1.1 \cdot M_{girder} + 1.3 \cdot M_{pelat}) \cdot y_{stop}}{I_s} = 184.84 \text{ MPa} \]
   Lateral bending stress values are assumed to be \( f_1 = 0 \)

2. Check the nominal melting resistance of the upper wing
   Check against the requirements 6.10.3.2.1
   Tension on the upper wing
   \[ f_{bu} = \sigma_{topflange} = 184.84 \text{ MPa} \]
   Reduction factor for bending
   \[ \phi_f = 0.90 \]
   For cross section with material a kind, \( Rh \) is taken 1,
   \[ Rh = 1 \]
   Nominal melting wing based on AASHTO 2017 LRFD (6.10.3.2.1-1)
   \[ \phi_f \cdot Rh \cdot F_y = 310.5 \text{ MPa} \]
   \[ f_{bu} + F_1 \leq \phi_f \cdot Rh \cdot F_y \]
   \[ 184.84 + 1 \leq 310.5 \text{ MPa} \ldots \text{OK} \]
   Ratio:
   \[ \frac{f_{bu} + f_1}{\phi_f \cdot Rh \cdot F_y} = 0.599 \]

3. Check Bending Resistance To Upper Flens
   Local buckling prisoners
   Calculate the ratio of the tension of the wing presses to the space according to AASHTO LRFD 2017 (6.10.8.2.2-3)
\[ \lambda_f = \frac{br}{2tf} = \frac{0.300}{0.02} = 7.5 \]

Calculate the limit of the slope to plate ratio
Compact compressive wing to suit AASHTO LRFD 2017 (6.10.8.2.2-4)
\[ \lambda_{pf} = 0.38 \sqrt{\frac{E}{f_y}} = 9.149 \]

If \( \lambda_f \leq \lambda_{pf} \)
7.5 \( \leq \) 9.149 then local buckling prisoners from upper wing is
\[ F_{nc,FLB} = R_b R_h f_y \]

Calculate the limit of slenderness ratio for plates noncompact body
\[ \lambda_{rw} = 5.7 \sqrt{\frac{E}{f_y}} = 137.24 \]

Calculate the value of the web loading shedding factor \( (R_b) \)
Factor \( R_b = 1 \) for strength checking when constructibility and if conditions the following are met:
- For a positive bending composite cross section without a longitudinal stiffner that meets the following requirements:
  \[ \frac{D}{t_w} \leq 150 \]
  \[ \frac{1760}{14} = 125.71 \leq 150 \quad \text{OK} \]

Then the local buckling resistance in the upper wing according to AASHTO LRFD 2017 (6.10.8.2.2-1) is:
\[ F_{nc,FLB} = R_b R_h f_{yc} = 345 \text{ MPa} \]

- Lateral torque bend resistance
  \( L_b = 5 \text{ m} \)
  Effective grating radius for lateral torsion (rt) bending according to AASHTO LRFD 2017 (6.10.8.2.3-9)
  \[ r_t = \frac{bf}{12 \left( 1 + \frac{D_{rw}}{bf} \right)} = \frac{0.300}{12 \left( 1 + \frac{1760}{0.02} \right)} = 0.081 \text{ m} \]
  Length without bracing (Lp) according to AASHTO LRFD 2017 (6.10.8.2.3-4)
  \[ L_p = 1.0 \cdot r_t \cdot \sqrt{\frac{E}{f_y}} = 1.0 \cdot 0.081 \cdot \sqrt{\frac{200000}{345}} = 1.950 \text{ m} \]
  Length without bracing (Lr) according to AASHTO LRFD 2017 (6.10.8.2.3-5)
  \[ L_r = \pi r_t \cdot \sqrt{\frac{E}{f_y}} = 3.14 \cdot 0.081 \cdot \sqrt{\frac{200000}{345}} = 6.052 \text{ m} \]
  If \( L_p < L_b < L_r \) then:
  \( C_b = 1 \)
  \[ F_{fr} = 0.7 \cdot F_y = 0.7 \cdot 345 = 241.5 \text{ MPa} \]
  \[ F_{nc,LTB} = C_b \left[ 1 - \left( 1 - \frac{F_{fr}}{R_h \cdot f_{yc}} \right) \left( \frac{L_{b} - L_{p}}{L_{r} - L_{p}} \right) \right] \]
  \[ = 1 \left[ 1 - \left( 1 - \frac{2415}{345} \right) \left( \frac{6.052 - 1.950}{6.052 - 1.950} \right) \right] \]
  \[ = 343.76 \leq 345 \quad \text{OK} \]

Use the smallest \( F_{nc} \) value from 2 conditions, so that:
\[ F_{nc} = \min (F_{nc,FLB}, F_{nc,LTB}) = 343.76 \text{ MPa} \]
\[ F_{bu} = 184.84 \text{ MPa} \]
\[ F_{bu} + \frac{1}{3} f_1 = 184.84 \text{ MPa} \]
\[ \varnothing_f, F_{nc} = 0.90 \times 343.76 \text{ MPa} = 309.38 \text{ MPa} \]
\[ F_{bu} + 1/3, f_1 \leq \varnothing_f, F_{nc} \]
184.84 MPa ≤ 309.38 MPa ................ OK

\[
\text{Ratio} = \frac{f_{bu} + f_1}{\sigma_{f, Fnc}} = \frac{184.84 + 0}{309.38} = 0.597
\]

4. Bottom Flens

\( y_{s_{bot}} = 0.9 \text{ m} \)

\[
\sigma_{\text{botflange}} = (1.1 \text{ Mgirder} + 1.3 \text{ Mpelat}) \cdot y_{s_{bot}} = 184.84 \text{ MPa}
\]

\( F_{bu} = \sigma_{\text{botflange}} = 184.84 \text{ MPa} \)

The tensile wing plate must meet the following equation:

\[
f_{bu} + f_1 \leq \sigma_{f, Fy}
\]

\( \sigma_{f, Fy} = 0.90 \cdot 345 = 310.5 \text{ MPa} \)

\( F_{bu} \leq \sigma_{f, Fy} \)

184.84 MPa ≤ 310.5 MPa……………OK

\[
\text{Ratio} = \frac{f_{bu}}{\sigma_{f, Fy}} = \frac{184.84}{310.5} = 0.595
\]

5. Body Plates (web)

To ensure that bending does not occur on the web during the construction process the requirements in equation 6.10.3.2.1-3 must be met,

\( F_{bu} \leq \sigma_{F_{crw}} \)

Bend the bending coefficient

\[
K = \frac{9}{(\frac{D}{D_c})^2} = \frac{9}{(\frac{880}{1760})^2} = 36
\]

Prisoners bend in the body

\[
F_{crw} = \frac{0.95 \cdot E_s \cdot k}{(\frac{D}{D_c})^2} = \frac{0.95 \cdot 200000 \cdot 36}{(\frac{14}{14})^2} = 432.80
\]

However, the \( F_{crw} \) value cannot be more big from:

\( F_{ye} = F_y = 345 \text{ MPa} \)

\( F_{yw} = F_y = 345 \text{ MPa} \)

\( R_h \cdot f_{ye} = 345 \text{ MPa} \)

\( f_{yw} \cdot 0.7 = 492.857 \text{ MPa} \)

then the bending prisoners in the body are:

\[
F_{crw} = \min(f_{crw}, R_h \cdot f_{yw} \cdot f_{yw} \cdot 0.7) = 345 \text{ MPa}
\]

\( \sigma_{f, Fy} = 0.90 \times 345 = 310.5 \text{ MPa} \)

Check bending resistance in the body

\( F_{bu} \leq \sigma_{f, Fy} \)

184.84 MPa ≤ 310.5 MPa …………… OK

\[
\text{Ratio} = \frac{f_{bu}}{\sigma_{f, Fy}} = 0.595
\]

In this study, the examination of the cross-section boundary conditions due to shear, service boundary conditions and fatigue boundary conditions are not included or not calculated. And go directly to the calculation phase of the strong boundary conditions.

**Strong Boundary Conditions**

*Due to Flexure*

1. Determination of Plastic Neutral Axes Based on Table D.6.1.1 AASHTO LRFD 2017

Compressed wing width \( b_{cf} = 300 \text{ mm} \)

Compressed wing thickness \( t_{cf} = 20 \text{ mm} \)

Wing width \( D_h = 300 \text{ mm} \)

Wing thickness \( t_h = 20 \text{ mm} \)

Web height \( D_w = 1760 \text{ mm} \)

Web thickness \( t_w = 14 \text{ mm} \)

Force plate press plate \( P_s = 0.85 \cdot f_c \cdot b_{cf} \cdot h_s = 1700 \text{ kN} \)

Axial force on the reinforcement of the deck plate \( P_r = 0 \text{ kN} \)
Axial force on the reinforcement under the deck plate \( P_{rb} = 0 \) kN

Axial force on the upper wing \( P_C = B_{ct} \cdot t_{cf} \cdot f_y = 2484 \) kN

Axial force on the web \( P_w = D_w \cdot t_w \cdot f_y = 8500.5 \) kN

Axial force on the lower wing \( P_t = b_{rt} t_{tf} f_y = 2070 \) kN

Case I
\[ P_t + P_w = 10570.5 \text{ kN} \]
\[ P_c + P_s + P_{rb} + P_{rt} = 4184 \text{ kN} \]

Case II
\[ P_t + P_w + P_c = 13054.5 \text{ kN} \]
\[ P_s + P_{rb} + P_{rt} = 1700 \text{ kN} \]

Because Case II meets the requirements, the PNA is in the upper wing, so:
\[ Y = \left( \frac{t_{cf}}{2} \right) \cdot \left( \frac{P_w + P_t - P_s - P_{rt} - P_{rb}}{P_c} + 1 \right) \]
\[ Y = 89.9 \text{ mm} \]

2. Check ductility according to AASHTO LRFD 2017 6.10.7.3-1
\[ D_p \leq 0.42 D_t \]
Distance from the top edge of the concrete deck to the neutral axis of the composite cross section at a plastic moment (\( D_p \))
\[ D_p = h_s + t_{cf} + Y = 129.9 \text{ mm} \]
Total height of composite cross section (\( D_t \))
\[ D_t = D + h_s = 1.82 \text{ m} \]
\[ 0.42 D_t = 764.4 \text{ mm} \]
\[ 129.9 \leq 764.4 \text{ ... OK} \]

3. Check steel compact section according to AASHTO LRFD 2017 6.10.7.1.2-1
\[ D_p \leq 0.1 D_t \]
\[ 0.1 D_t = 182 \text{ mm} \]
\[ 182 \leq 764.4 \text{ ... OK} \]

4. Calculation of AASHTO LRFD 2017 plastic moment D6.1-2
\[ D_t = t_{cf} + D_w + \frac{t_f}{2} - Y = 1700.1 \text{ mm} \]
\[ d_s = h_s/2 + t_{cf} + Y = 119.9 \text{ mm} \]
\[ d_w = D_w/2 + t_f - Y = 810.1 \text{ mm} \]
\[ d_{rt} = 0 \text{ mm} \]
\[ d_{rb} = 0 \text{ mm} \]

So that the plastic moment can be calculated by:
\[ M_p = \frac{P_c}{2 \cdot t_{cf}} \left[ Y^2 - (t_{cf} - Y)^2 \right] + P_s \cdot d_s + P_{rt} \cdot d_{rt} + P_{rb} \cdot d_{rb} + P_w \cdot d_w + P_t \cdot d_t \]
\[ M_p = 7859.652 \text{ kN} \cdot \text{m} \]

Calculate the nominal moment value
\[ M_n = M_p \left( 1.07 - 0.7 \cdot \frac{D_p}{D_t} \right) \]
\[ M_n = 8016.843 \text{ kN} \cdot \text{m} \]
\[ M_u = 3595.38 \text{ kN} \cdot \text{m} \]
\[ \Theta_f = 1.0 \]
\[ \Theta_f \cdot M_n = 8016.843 \text{ kN} \cdot \text{m} \]

5. Check the cross-section capacity according to AASHTO LRFD 2017 6.11.7.1.1-1
\[ M_u \leq \Theta_f \cdot M_n \]
\[ 3595.38 \leq 8016.843 \text{ kN} \cdot \text{m} \text{ ... OK} \]

Ratio = \[ \frac{M_u}{\Theta_f \cdot M_n} = 0.449 \]
Design of Non-Composite Girder Plate Based on SNI 1729: 2015
Components of Profile Structure I Double Symmetrical And Single Symmetrical With A Curved Slim Body On Major Axis (F5 - SNI 1729: 2015)
The nominal flexural strength $M_n$, must be the lowest value obtained in accordance with the boundary conditions of the compressed wing melting, torsion-lateral bending, localized compressive wing bending and tensile wing melting.

1. Melt Wings Press
$M_n = F_y S_{xt}$
Looking for $R_{pg}$:

\[
R_{pg} = 1 - \frac{a_w}{1200 + 300a_w} \left( \frac{h_c}{t_w} - 5.7 \frac{E}{F_y} \right) \leq 1.0
\]

Meanwhile, to search for $a_w$ itself

\[
a_w = \frac{h_c \cdot t_w}{b_{fc} \cdot t_{fc}} = 4.2
\]

\[
R_{pg} = 1 - \frac{4.2}{1200 + 300 \cdot 4.2} \left( \frac{1800}{300} - 5.7 \frac{20000}{345} \right) \leq 1.0
\]

$R_{pg} = -8.68 \leq 1.0$
Maka $M_n = R_{pg} F_y S_{xc}$
$M_n = -8.68 \cdot 345.176288948$
$M_n = 52.79 \times 10^{10} \text{Nmm} = 52791.49 \text{kNm}$

2. Bend Lateral Torque
For cross sections with compact wings, the boundary state of the local bending of the compressed wing should not be used. Therefore in this study for compact wing cross sections it should not be used.

3. Bend Local Wing Press
$M_n = R_{pg} F_{cr} S_{xc}$
Look for the $F_{cr}$ value to bend the local wing press:

- $F_{cr} = \left[ F_y - (0.35F_y) \left( \frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right]$

- $\lambda = \frac{b_{fc}}{2t_{fc}} = \frac{300}{220} = 1.36$
- $\lambda_{pf} = \lambda_p = 0.38 \frac{E}{F_{yc}} = 9.15$
- $\lambda_{rf} = \lambda_r = 0.95 \frac{K_c E}{F_L}$

$K_c = \frac{4}{\sqrt{h/t_w}} = 0.36$
$F_L = 0.7 \cdot F_y = 241.5$

\[
\lambda_{rf} = \lambda_r = 0.95 \frac{K_c E}{F_L} = 16.40
\]

So,

\[
F_{cr} = \left[ F_y - (0.35F_y) \left( \frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] = 321.44
\]

So that,

$M_n = R_{pg} F_{cr} S_{xc}$
$M_n = 49186365278.36 \text{ Nmm}$
4. Melt Wing Pull
\[ M_n = f_y S_{xt} \]
\[ M_n = 345 \times 17628894.8 \]
\[ M_n = 6081968706 \text{ Nmm} \]
\[ M_n = 6081.97 \text{ kNm} \]

From the results of the whole calculation, the smallest \( M_n \) value is taken, the \( M_n \) value which is in the tensile wing melting of 6081.97 kNm.

Comparison of Similarities and Differences

Below is a table of comparative results between the AASHTO LRFD Bridge Design Specification 2017 code with the SNI 1729: 2015 code in the planning of bending non-composite plate girder bridge design:

| No | Code                                           | \( M_n \)       | Satuan | \( \phi M_n \) | Satuan | Status |
|----|------------------------------------------------|----------------|--------|----------------|--------|--------|
| 1  | AASHTO LRFD Bridge Design Specifications 2017 | 3595.38 kNm     |         | 8016.84 kNm    |        | OK     |
| 2  | SNI 1729:2015                                  | 3595.38 kNm     |         | 6081.97 kNm    |        | OK     |

CLOSING

In this chapter, the authors provide conclusions and suggestions relating to the results that the authors examined. The conclusion was obtained from the results of the analysis and interpretation of existing data. Meanwhile, suggestions are given as reference material in subsequent studies.

CONCLUSIONS

From the comparative studies that have been made, some conclusions can be drawn, as follows:

1. In planning the plate girder bending using AASHTO LRFD Bridge Design Specifications 2017 gives a greater \( \phi M_n \) value so that a smaller profile dimension can be used than the results of planning using SNI 1729: 2015.
2. On the contrary, planning for plate girder bending using SNI 1729: 2015 results in a smaller \( \phi M_n \) value so that the dimensions of the profile used are greater than the results of planning using AASHTO LRFD Bridge Design Specifications 2017.
3. The results of planning the flexible plate girder using the AASHTO LRFD Bridge Design Specifications 2017 regulations are much safer when compared to the results of the design using SNI 1729: 2015 regulations.

From the values obtained it can be concluded that the two regulations namely AASHTO LRFD Bridge Design Specifications 2017 and SNI 1729: 2015 in the stability planning of non-composite plate girder bridges produce a safe and strong plan because they meet the applicable provisions namely Moment (\( Mu <\phi M_n \)).

Suggestions

From the comparative studies that have been carried out, several things that can be developed in further research are as follows:

1. It is necessary to compare non-composite bridge planning based on AASHTO LRFD Bridge Design Specifications 2017 and SNI 1729: 2015 for Constructability, Serviceability, Fatigue and Fracture.
2. It is necessary to compare the planned budget of the design results of the non-composite plate girder bridge design based on AASHTO LRFD Bridge Design Specifications 2017 and SNI 1729: 2015.

REFERENCES

[1] Bakosurtanal. Badan Informasi Geospasial. Bakosurtanal. [Online] 2014. http://www.bakosurtanal.go.id.
[2] Supriyadi, Bambang and Muntohar, A. S. Jembatan. Yogyakarta : Beta Offset, 2007.
[3] AASHTO. AASHTO LRFD Bridge Design Specifications 8th Edition. Washington, DC : American Association of State Highway and Transportation Officials, 2017.
[4] Indonesia, Standar Nasional. SNI 1729:2015 Spesifikasi untuk Bangunan Gedung Baja Struktural. Jakarta : Badan Standardisasi Nasional, 2015.
[5] AISC. An American National Standard ANSI/AISC 360-10 : Load Specification for Structural Steel Buildings. Chicago : American Institute of Steel Construction, Inc., 2010.
[6] Dewobroto, W. Struktur Baja Perilaku, Analisis & Desain–AISC 2010 Edisi ke-2. Tanggerang : Penerbit Jurusan Teknik Sipil UPH, 2016.
[7] Setiawan, A. Perencanaan Struktur Baja Dengan Metode LRFD. Semarang : Erlangga, 2008.
[8] Indonesia, Standar Nasional. SNI 1725:2016 Pembebanan Untuk Jembatan. Jakarta : Badan Standardisasi Nasional, 2016.
[9] Situstekniksipil.com. Klasifikasi Jembatan. Situstekniksipil. [Online] 23 October 2017. https://www.situstekniksipil.com/2017/10/klasifikasi-jembatan.html.
[10] Testindo.com. Penjelasan Konstruksi Jembatan, Lengkap!!! Testindo.com. [Online] 2 July 2018. https://testindo.com/article/359/konstruksi-jembatan.
[11] Essen, Donald. Sesi 06 – Perencanaan Struktur Baja Berdasarkan AASHTO LRFD Bridge Design Specifications 8th Edition. Pertamina University, Jakarta, Jakarta, Indonesia : s.n., 2-5 September 2019