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

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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 ($ØM_n$) of 8016.843 kNm for AASHTO LRFD Bridge Design Specifications 2017 and Nominal Moment value ($ØM_n$) 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 <$ØM_n$).

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.

Fig. 1 Application Of Using Plate Girder Bridges On Japek II Elevated Toll Road

Source: Data in research, 2019

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.

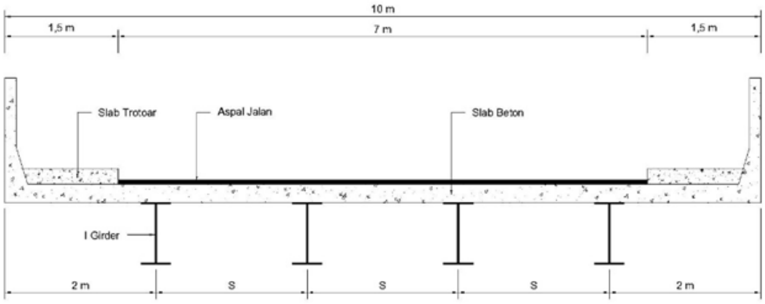
In the design of this steel frame bridge, it will use two bridge structure design regulations, namely using the AASHTO LRFD Bridge Design Specifications (American) and SNI (Indonesia), which in the end can be seen differences and similarities in the results of the design. The two construction regulations 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

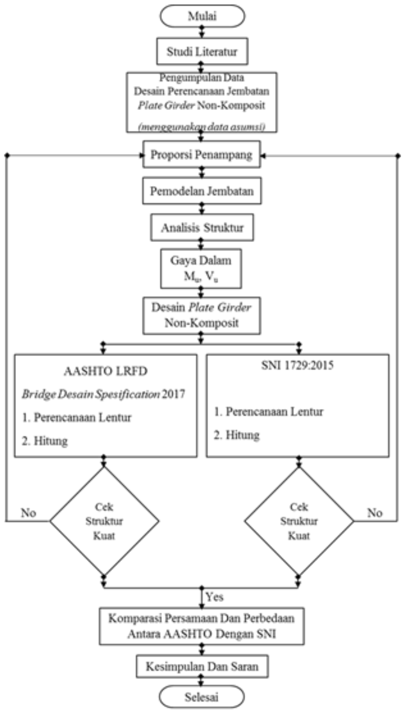


Source: Data in research, 2019

Assumption Technical Data

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

Fig. 3 Research Flow Chart



Source: Data in research, 2019

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 Ysteel = 78.50 kN/m^3

1.2 Deck Weight

Deck has dimensions:
cross-sectional area A_{deck} = 0.4 m^2
Concrete specific gravity $\Upsilon_{concrete}$ = 25 kN/m^3
Then the deck load = A_{deck} x Υ_{beton} =10 kN/m

1.3 Diaphragm Weight
Diaphragm has dimensions:
Diaphragm volume V = 0.006096 m^3
Steel specific gravity Υ_{steel} = 78.5 kN / m^3

2. Additional Dead Load / Utilities (MA)

2.1 Load Pavement
Concrete specific gravity $\Upsilon_{concrete}$ = 25 kN/m^3
wide sidewalk $A_{sidewalk}$ = 0.189 m2
Load of sidewalk $q_{sidewalks}$ =25 x 0.198=4.73 kN/m (per side)

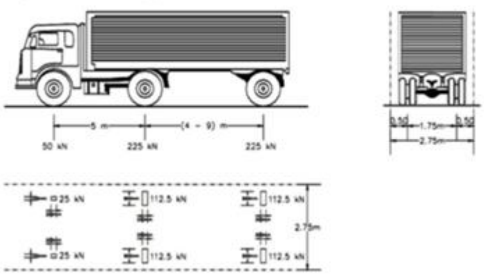
2.2 Ralling Loads
Concrete specific gravity $\Upsilon_{concrete}$ = 25 kN/m^3
Wide railing concrete A_{rall} = 0.196 m2
Load concrete railing q_{rall} = 25 x 0.196= 4.90 kN / m (per side)

2.3 Asphalt Loads
Asphalt density $\Upsilon_{asphalt}$ = 22.4 kN / m^3
Asphalt thickness $t_{asphalt}$= 0.05 m
Road width b_{road}= 7 m
Asphalt loads $q_{asphalt}$ = 22.4 x 0.05 x 7
= 7.84 kN / m (per entire bridge width)

2.4 Concrete Screed Loads
Concrete specific gravity $\Upsilon_{concrete}$ = 25 kN/m^3
Concrete screed area A_{screed} = 0.35 m^3
Concrete screed load q_{screed} = 25 x 0.35 = 8.75 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 kN/m

3. "T" Truck Load Calculation (TT)
The truck load 'T' used is in accordance with SNI 1725: 2016[8] as shown

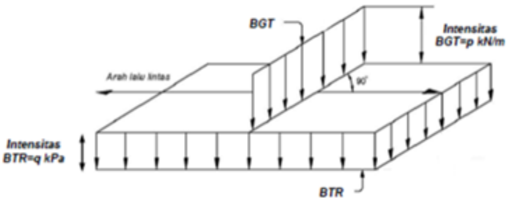
Fig. 4 Truck Load 'T'



Dynamic load factor (FBD) for BGT is taken FBD = 0.300
Front wheel weight P_{rd} = 25,000 kN
Rear wheel weight P_{rb} = 112,500 kN
Weight of front wheel + FBD P_1 = 32.5 kN
Rear wheel weight + FBD P_2= 146.25 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 ≤ 30 m → q = 9.0 kPa
If L > 30 m → q = 9.0 $(0.5 + \frac{15}{L})$ kPa

Fig. 5 Lane Load 'D'



Lane density factors must be calculated according to Table 3.1

Table 3.1 Lane Density Factors

Jumlah lajur yang dibebani	faktor kepadatan lajur
1	1,2
≥ 2	1

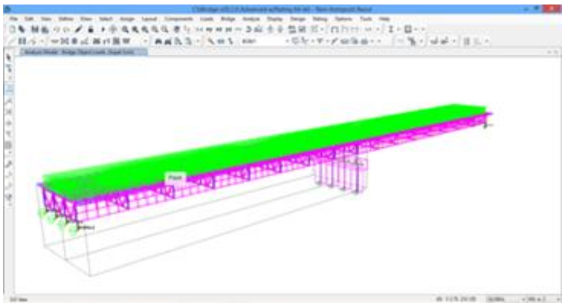
Evenly distributed load intensity (BTR) q = 7,875 KPa = kN/m^2
Centralized line load intensity (BGT) p = 49,000
Dynamic load factor (FBD) for BGT is taken FBD = 0.300
Load evenly distributed (BTR) q = 14.40 kN/m
Central line load (BGT) + (FBD) p = 109.76 kN/m

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 m
   b. Intensity of pedestrian loads PTP = 5.00 kN/m^2

Application Load On Steel Girder Composite Modeling

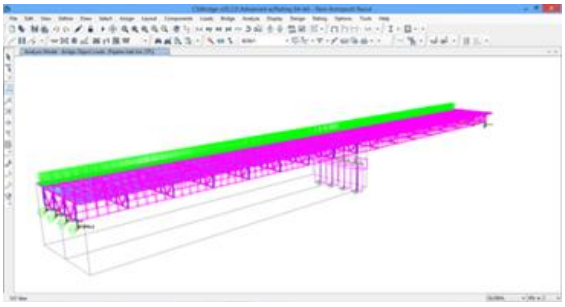
1. Application load of asphalt on the bridge

Fig 6. Modeling loading asphalt 5 cm



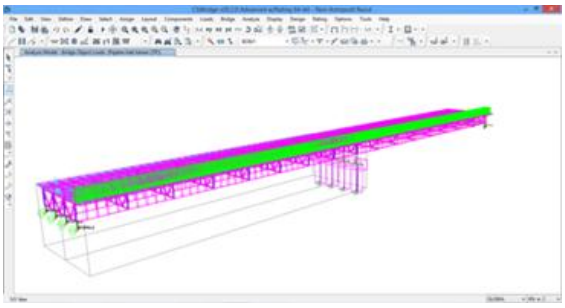
2. Application load on the left side of the pedestrian bridge

Fig. 7 Modeling 3D pedestrian loading Left



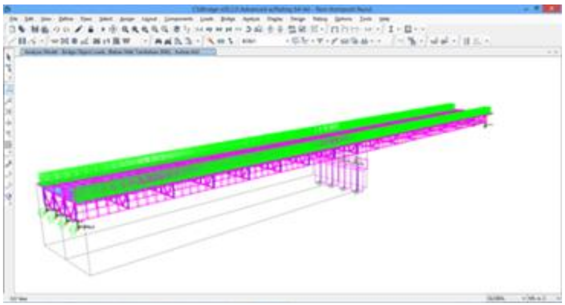
3. Application load on the right side of the pedestrian bridge

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

Keadaan Batas	MS MA TA PR PL SH	TT TD TB TR TP	EU	EW_s	EW_L	BF	EU_n	TG	ES	Gunakan salah satu		
										EQ	TC	TV
Kuat I	γ_p	1,8	1,00	-	-	1,00	0,50/1,20	γ_{TG}	γ_{ES}	-	-	-
Kuat II	γ_p	1,4	1,00	-	-	1,00	0,50/1,20	γ_{TG}	γ_{ES}	-	-	-
Kuat III	γ_p	-	1,00	1,40	-	1,00	0,50/1,20	γ_{TG}	γ_{ES}	-	-	-
Kuat IV	γ_p	-	1,00	-	-	1,00	0,50/1,20	-	-	-	-	-
Kuat V	γ_p	-	1,00	0,40	1,00	1,00	0,50/1,20	γ_{TG}	γ_{ES}	-	-	-
Ekstrem I	γ_p	γ_{EQ}	1,00	-	-	1,00	-	-	-	1,00	-	-
Ekstrem II	γ_p	0,50	1,00	-	-	1,00	-	-	-	-	1,00	1,00
Daya layan I	1,00	1,00	1,00	0,30	1,00	1,00	1,00/1,20	γ_{TG}	γ_{ES}	-	-	-
Daya layan II	1,00	1,30	1,00	-	-	1,00	1,00/1,20	-	-	-	-	-
Daya layan III	1,00	0,80	1,00	-	-	1,00	1,00/1,20	γ_{TG}	γ_{ES}	-	-	-
Daya layan IV	1,00	-	1,00	0,70	-	1,00	1,00/1,20	-	1,00	-	-	-
Fatik (TD dan TR)	-	0,75	-	-	-	-	-	-	-	-	-	-

Catatan : - γ_p dapat berupa $\gamma_{MS}, \gamma_{MA}, \gamma_{TA}, \gamma_{PR}, \gamma_{PL}, \gamma_{SH}$ tergantung beban yang ditinjau
- γ_{EQ} adalah faktor beban hidup kondisi gempa

Cross Section Proportion

Determining the Cross-Section Proportion

$$\frac{d}{L} = \frac{1}{25}$$

$$d = \frac{L}{30} = \frac{40000\ mm}{25} = 1600\ mm$$

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

$d + (dx10\%) = 1600 + (1600x10\%) = 11760\ mm$

Then take grades $d = 1800\ mm$

$t_s = 250\ mm$

Take S = 2.5 m & S' = 0.75

Flange width is controlled when handling & transportation

$b_{fc} \geq \frac{L}{85}\ mm$ → L= *shipping length* ≈ 12 m

$b_{fc} \geq \frac{12000}{85} = 142\ mm \geq 300\ mm \rightarrow b_{fc} \geq 300\ mm$

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:

$b_f \geq \frac{D}{6} = \frac{1760}{6} = 293.33\ mm$

Use b_f = 350 for the upper and lower flanges

$\frac{b_f}{2t_f} \leq 9.2 = \frac{350}{2x20} = 8.75 < 9.2$ (top flens NSBA)

$\frac{b_f}{2t_f} \leq 9.2 = \frac{350}{2x20} = 8.75 < 9.2$ (bottom flens 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 used 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

1. Proportion of Body Plate Without Stiffener
   Calculated as follows:
   $\frac{D_w}{t_w} = \frac{1760}{14} = 125.71\ mm$
   Check web stability = $\frac{D_w}{t_w} \leq 150 \rightarrow 125.71 \leq 150$, 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
     $\frac{b_f}{2t_f} \leq 12 \rightarrow \frac{300}{2.20} = 7.5 \leq 12 \ldots$*OK*
   - AASHTO LRFD 2017 6.10.2.2-2
     $b_f \geq \frac{D_w}{6} \rightarrow \frac{1760}{6} = 293.33mm \rightarrow 0.3m \geq 0.293m \ldots OK$
   - AASHTO LRFD 2017 6.10.2.2-3
     $t_f \geq 1.1t_w \rightarrow 1.1t_w = 0.015m$
     $t_f = 0.020m$
     $0.020 \geq 0.015 \ldots$*OK*
   - AASHTO LRFD 2017 6.10.2.2-4

$$0.1 \leq \frac{I_{yc}}{I_{yt}} \leq 10$$

$$I_{yc} = \frac{t_f . b_f^3}{12} = \frac{0.020 . 0.0252}{12} = 0.000042\, m^4$$

$$I_{yt} = \frac{t_f . b_f^3}{12} = \frac{0.020 . 0.0252}{12} = 0.000042\, m^4$$

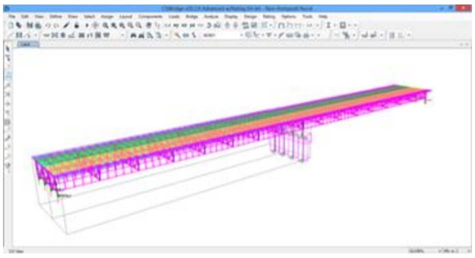
$$0.1 \leq \frac{I_{yc}}{I_{yt}} \leq 10$$

$$0.1 \leq 1 \leq 10$$

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.

Fig. 10 3D Modeling Non-composite bridge



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

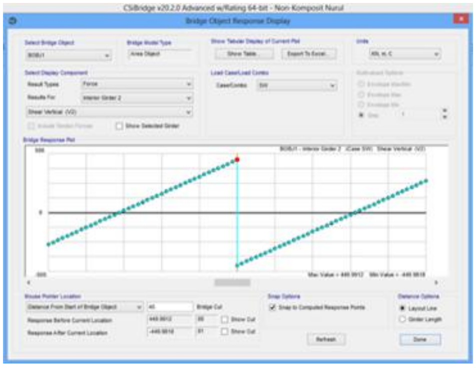
Fig. 11 Field Moment Non-Composite Steel Structures

The value of internal forces in the interior girder 1
M1 = -3595.38
The value of internal forces in the interior girder 2
M2 = -3595.25
The value of internal forces in the interior beam 1
M1 = -3593.4
The value of internal forces in the interior beam 2
M2 = -3593.32
The value of internal forces in the interior slab 1
M1 = -0.0118
The value of internal forces in the interior slab 2
M2 = -0.0118

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
The value of the shear force on the interior of the girder 2
V2 = -449.9818
The value of the shear force on the interior beam 1
V1 = -442.4905
The value of the shear force on the interior beam 2
V2 = -442.4755
The value of the shear force on the interior of the slab 1
V1 = -7.5063
The value of the shear force on the interior of the slab 2
V2 = -7.5063
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

Fig. 13 Non Composite Elastic Flexural Section Properties

Non Composite Elastic Flexural Section Properties					
About - X					
Part	A	y	A.y	A x (y-Yb)^2	Iox
Top Flange	6000.0	1790.0	10740000.0	4752600000.0	200000.0
Web	24640.0	900.0	22176000.0	0.0	6360405333.3
Bottom Flange	6000.0	10.0	60000.0	4752600000.0	200000.0
Cover plate	0.0	0.0	0.0	0.0	0.0
Long Stiff 1	0.0	0.0	0.0	0.0	0.0
Long Stiff 2	0.0	0.0	0.0	0.0	0.0
	36640.0		32976000.0	9505200000.0	6360805333
	A_g	36640.0	mm^2		
	$y_{c.g.c}$	900.0	mm		
	I_x	15866005333.3	mm^4		
About - Y					
Part	A	x	A.x	A.(x-X)^2	Ioy
Top Flange	6000.0	0.0	0.0	0.0	45000000.0
Web	24640.0	0.0	0.0	0.0	402453.3
Bottom Flange	6000.0	0.0	0.0	0.0	45000000.0
Cover plate	0.0	0.0	0.0	0.0	0.0
Long Stiff 1	0.0	0.0	0.0	0.0	0.0
Long Stiff 2	0.0	0.0	0.0	0.0	0.0
	36640.0		0.0	0.0	90402453.3
	A_g	36640.0	mm^2		
	$x_{c.g.c}$	0.0	mm		
	I_y	90402453.3	mm^4		

Non Composite Elastic Torsional Section Properties

Fig. 14 Non Composite Elastic Torsional Section Properties

Non Composite Elastic Torsional Section Properties					
Cover Plate and Bottom Flange Does Not Act Compositely					
Part	b	t	α	α.b.t³	(α.b.t³)/3
Top Flange	300.0	20.0	1.0	2400000.0	800000.0
Web	1780.0	14.0	1.0	4884320.0	1628106.7
Bottom Flange	300.0	20.0	1.0	2400000.0	800000.0
Cover plate	0.0	0.0	0.0	0.0	0.0
Long Stiff 1 Left	0.0	0.0	0.0	0.0	0.0
Long Stiff 1 Right	0.0	0.0	0.0	0.0	0.0
Long Stiff 2 Left	0.0	0.0	0.0	0.0	0.0
Long Stiff 2 Right	0.0	0.0	0.0	0.0	0.0
					3228106.7
	J	3228106.7	mm⁴		
Cover Plate and Bottom Flange Act Compositely					
Part	b	t	α	α.b.t³	(α.b.t³)/3
Top Flange	300.0	20.0	1.0	2400000.0	800000.0
Web	1780.0	14.0	1.0	4884320.0	1628106.7
Bottom Flange	300.0	20.0	1.0	2400000.0	800000.0
Long Stiff 1 Left	0.0	0.0	0.0	0.0	0.0
Long Stiff 1 Right	0.0	0.0	0.0	0.0	0.0
Long Stiff 2 Left	0.0	0.0	0.0	0.0	0.0
Long Stiff 2 Right	0.0	0.0	0.0	0.0	0.0
					3228106.7
	J	3228106.7	mm⁴		

bf

Girder moment of inertia about x = Σ (A x $(y - Yb)^2$ + Iox

=9505200000.0+6360805333

= 15866005333.3 mm^4

Girder moment of inertia about y = Σ (A x $(x - X)^2$ + Ioy = 0 + 90402453.3

= 90402453.3 mm^4

Girder top elastic section modulus about x = $\frac{I_x}{(H+tcp-Yc.g.c)} = \frac{15866005333.3}{(1800+0-900)}$

= 17628894.8 mm^3

Girder bottom elastic section modulus about x = $\frac{I_x}{(H+tcp-Yc.g.c)}$

= $\frac{15866005333.3}{(1800+0-900)}$

= 17628894.8 mm^3

Girder left elastic section modulus about y = $\frac{I_y}{(B_{ft}/2)} = \frac{90402453.3}{(300/2)}$

= 602683.0 mm^3

Girder right elastic section modulus about y = $\frac{I_y}{(B_{ft}/2)} = \frac{90402453.3}{(300/2)}$

= 602683.0 mm^3

Girder plastic section modulus about x = $\frac{Mp_x}{f_y \,.\, 10^6} = \frac{7859.65}{345 \,.\, 10^6}$

= 22781600 mm^3

Girder plastic section modulus about y = $\frac{Mp_y}{f_y \,.\, 10^6} = \frac{340.25}{345 \,.\, 10^6}$

= 986240mm^3

Radius of gyration about x = $\sqrt[2]{\frac{I_x}{A_g}} = \sqrt[2]{\frac{15866005333.3}{36640}}$

= 658 mm

Radius of gyration about y = $\sqrt[2]{\frac{I_y}{A_g}} = \sqrt[2]{\frac{90402453.3}{36640}}$

= 49.7 mm

Centroid from bottom = $\frac{\Sigma A.y}{\Sigma A} = \frac{32976000}{36640}$ = 900 mm

Centroid from mid of web= $\frac{\Sigma A.x}{\Sigma A} = \frac{0}{36640}$ = 0 mm

Torsional Constant = $\sum \frac{(\alpha.b.t^3)}{3} = \frac{9{,}684{,}320.00}{3}$ = 3228106.7 mm

Warping Constant = $\sum \frac{(\alpha.b.t^3)}{3} = \frac{9{,}684{,}320.00}{3}$ = 71289000000000 mm

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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)

$f_{yf} = f_y = 345$ MPa

$f_{yf} \leq 485$ MPa

$345 \leq 485$ MPa....................OK

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

$2 \cdot \frac{D_{cp}}{t_w} \leq 3.76 \sqrt{\frac{E_s}{fc'}}$

Where:

ys_{top} = 0.59 m

$D_{cp} = ys_{top} - t_f$ = 0.57 m

So,

$2 \cdot \frac{D_{cp}}{t_w} = \frac{1140}{14} = 81.42$ mm

$3.76 \sqrt{\frac{E_s}{fc'}} = 265,87$

$81.42 \leq 265,87$................OK

Non-Composite Plate Girder Design Based on AASHTO LRFD Bridge Design Specification 2017[3]

Check the State of the Cross Section

During construction

Due to flexure

1. Upper Flens
   Style that works:
   M_{girder} = 3595.38 kN.m
   M_{plate} = 781.25 kN.m
   I_s = 15866005333.3 mm^4
   Tension on the upper wing:
   $\sigma_{topflange} = \frac{(1.1 \text{ M girder} + 1.3 \text{ M pelat}) \cdot \text{ystop}}{I_s} = 184.84$ MPa
   Lateral bending stress values are assumed to be f1= 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}= σtopflange = 184.84 MPa
   Reduction factor for bending
   $Ø_f = 0.90$
   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)
   $Ø_f \cdot Rh \cdot Fy = 310.5$ MPa
   $f_{bu} + F1 \leq Ø_f \cdot Rh \cdot Fy$
   184.84 + 1 ≤ 310.5 MPa .... OK
   Ratio = $\frac{f_{bu+f1}}{Ø_f \cdot Rh \cdot Fy} = 0.599$
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)

$\lambda_f = \frac{b_f}{2tf} = \frac{0,300}{2\ .\ 0,02} = 7.5$

Calculate the limit of the slope to plate ratio
Compact compressive wing to suit AASHTO LRFD 2017 (6.10.8.2.2-4)

$\lambda_{pf} = 0.38\sqrt{\frac{E_s}{f_y}} = 9.149$

If $\lambda_f \leq \lambda_{pf}$

7.5 ≤ 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

$\lambda_{rw} = 5.7\sqrt{\frac{E_s}{f_y}} = 137.24$

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:
  $\frac{D}{t_w} \leq 150$
  $\frac{1760}{14} = 125.71 \leq 150$ .... OK
  Then the local buckling resistance in the upper wing according to AASHTO LRFD 2017 (6.10.8.2.2-1) is:
  $F_{nc\ FLB} = R_b R_h f_{yc} = 345$ MPa
- Lateral torque bend resistance
  L_b = 5 m
  Effective grating radius for lateral torsion (rt) bending according to AASHTO LRFD 2017 (6.10.8.2.3-9)
  $r_t = \frac{bf}{\sqrt{12\ (1+\frac{1}{3}.\frac{D.tw}{bf.tf}}} = \frac{0,300}{\sqrt{12\ (1+\frac{1}{3}.(\frac{1,760\ .\ 0,014}{0,300\ .\ 0,020}))}} = 0.081$ m
  Length without bracing (Lp) according to AASHTO LRFD 2017 (6.10.8.2.3-4)
  $L_p = 1.0\ .\ rt\ .\ \sqrt{\frac{E_s}{f_y}} = 1.0\ .\ 0.081\ .\ \sqrt{\frac{200000}{345}} = 1.950$ m
  Length without bracing (L_r) according to AASHTO LRFD 2017 (6.10.8.2.3-5)
  $L_r = \pi\ rt\ .\ \sqrt{\frac{E_s}{f_y}} = 3,14\ .\ 0,081\ .\ \sqrt{\frac{200000}{345}} = 6.052$ m
  If $L_p < L_b < L_r$ then:
  Cb = 1
  $F_{yr} = 0.7.\ F_y = 0.7.\ 345 = 241.5$ MPa
  $F_{nc\ LTB} = C_b\left[1-\left(1-\frac{Fyr}{Rh\ .\ Fyc}\right)\left(\frac{Lb-Lp}{Lr-Lp}\right)\right].\ R_h f_{yc} \leq R_b R_h f_{yc}$
  $=1\left[1-\left(1-\frac{241.5}{1\ .\ 345}\right)\left(\frac{2-1.950}{6.052-1.950}\right)\right].345 \leq 345$
  $= 343.76 \leq 345$ .... OK
  Use the smallest F_{nc} value from 2 conditions, so that:
  $F_{nc} = \min(F_{nc}FLB, F_{nc}LTB) = 343.76$ MPa
  F_{bu} = 184.84 MPa
  $F_{bu} + \frac{1}{3}.\ f1 = 184.84$ MPa
  $\text{Ø}_f.\ F_{nc} = 0.90 \times 343.76$ MPa = 309.38 MPa
  $F_{bu} + 1/3.\ f1 \leq \text{Ø}_f.\ F_{nc}$

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184.84 MPa ≤ 309.38 MPa .... OK

Ratio $= (\frac{f_{bu+\frac{1}{3}f1}}{Ø_f\,.\,Fnc}) = (\frac{184.84+0}{309.38}) =$ 0.597

4. Bottom Flens

ys_{bot} = 0.9 m

$\sigma_{botflange} = \frac{(1.1\ \text{Mgirder}+1.3\ \text{Mpelat})\,.ys_{bot}}{I_s}$ = 184.84 MPa

F_{bu} = $\sigma_{botflange}$ = 184.84 MPa

The tensile wing plate must meet the following equation:

$f_{bu} + f_1 \leq Ø_f\,.\,Rh\,.\,Fy$

$Ø_f\,.\,Rh\,.\,Fy$ = 0.90 . 1 . 345 = 310.5 MPa

$F_{bu} \leq Ø_f\,.\,Rh\,.\,Fy$

184.84 MPa ≤ 310.5 MPa.............OK

Ratio $\frac{f_{bu}}{Ø_f\,.Rh\,.Fy} = \frac{184.84}{310{,}5}$ = 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,

$F_{bu} \leq ØF_{crw}$

Bend the bending coefficient

$K = \frac{9}{(\frac{Dc}{D})^2} = \frac{9}{(\frac{880}{1760})^2} = 36$

Prisoners bend in the body

$F_{crw} = \frac{0.95\,.Es\,.k}{(\frac{D}{tw})^2} = \frac{0.95\,.200000\,.36}{(\frac{1760}{14})^2} = 432.80$

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

$F_{ye} = F_y$ = 345 MPa

$F_{yw} = F_y$ = 345 MPa

$R_h\,.\,f_{ye}$ = 345 MPa

$\frac{f_{yw}}{0.7}$ = 492.857 MPa

then the bending prisoners in the body are:

$F_{crw} = \min\,(f_{crw}, R_h\,.\,f_{ye}, \frac{f_{yw}}{0.7})$ = 345 MPa

$Ø_f\,.\,F_{crw}$ = 0.90 x 345 = 310.5 MPa

Check bending resistance in the body

$F_{bu} \leq Ø_f.F_{crw}$

184.84 MPa ≤ 310.5 MPa .... OK

Ratio $= \frac{f_{bu}}{Ø_f\,.F_{crw}}$ = 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 mm
Compressed wing thickness	t_{cf} = 20 mm
Wing width	b_{tf} = 300 mm
Wing thickness	t_{ft} = 20 mm
Web height	D_w = 1760 mm
Web thickness	t_w = 14 mm
Force plate press plate	$P_s = 0.85.f_{c'}.b_{ef}.h_s$ = 1700 kN
Axial force on the reinforcement of the deck plate	P_{rt} = 0 kN

Axial force on the reinforcement under the deck plate P_{rb} = 0 kN
Axial force on the upper wing $P_C = B_{cf}.\ t_{cf} \cdot f_y$ = 2484 kN
Axial force on the web $P_w = D_w.\ t_w \cdot f_y$ = 8500.5 kN
Axial force on the lower wing $P_t = b_{tf}.\ t_{tf} \cdot f_y$ = 2070 kN
Case I
$P_t + P_w$ = 10570.5 kN
$P_c + P_s + P_{rb} + P_{rt}$ = 4184 kN

Case II
$P_t + P_w + P_c$ = 13054.5 kN
$P_s + P_{rb} + P_{rt}$ = 1700 kN
Because Case II meets the requirements, the PNA is in the upper wing, so:

$$Y = \left(\frac{t_{cf}}{2}\right).\left(\frac{P_w + P_t - P_s - P_{rt} - P_{rb}}{P_c} + 1\right)$$

$Y = 89.9\ mm$

2. Check ductility according to AASHTO LRFD 2017 6.10.7.3-1
   $D_p \le 0.42D_t$
   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 mm
   Total height of composite cross section (D_t)
   $D_t = D + h_s$ = 1.82 m
   $0.42D_t$ = 764.4 mm
   $D_p \le 0.42D_t$
   129.9 ≤ 764.4 ... OK
3. Check steel compact section according to AASHTO LRFD 2017 6.10.7.1.2-1
   $D_p \le 0.1D_t$
   $0.1D_t$ = 182 mm
   182 ≤ 764.4 ... OK
4. Calculation of AASHTO LRFD 2017 plastic moment D6.1-2
   $D_t = t_{cf} + D_w + t_{tf}/2 - Y$ = 1700.1 mm
   $d_s = h_s/2 + t_{cf} + Y$ = 119.9 mm
   $d_w = D_w/2 + t_{tf} - Y$ = 810.1 mm
   d_{rt} = 0 mm
   d_{rb} = 0 mm
   So that the plastic moment can be calculated by:

$$M_p = \frac{P_c}{2.t_{cf}}.\left[Y^2 - \left(t_{cf} - Y\right)^2\right] + P_s.d_s + P_{rt}.d_{rt} + P_{rb}.d_{rb} + P_w.d_w + P_t.d_t$$

   $M_p = 7859.652\ kN.m$
   Calculate the nominal moment value

$$M_n = M_p.\left(1.07 - 0.7.\frac{D_p}{D_t}\right)$$

   $M_n = 8016.843\ kN.m$
   $M_u = 3595.38\ kN.m$
   $\emptyset_f = 1.0$
   $\emptyset_f\,.M_n = 8016.843\ kN.m$
5. Check the cross-section capacity according to AASHTO LRFD 2017 6.11.7.1.1-1
   $M_u \le \emptyset_f\,.M_n$
   $3595.38 \le 8016.843\ \mathrm{kN.m} \ldots$ OK
   Ratio = $\frac{M_u}{\emptyset_f\,.M_n} = 0.449$

Design of Non-Composite Girder Plate Based on SNI 1729: 2015[4]

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}:

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

Meanwhile, to search for a_w itself = $\frac{h_c \,.t_w}{b_{fc} \,.\, t_{fc}} = 4.2$

$$R_{pg} = 1 - \frac{4.2}{1200 + 300\,.\,4.2}\left(\frac{1800}{300} - 5.7\sqrt{\frac{20000}{345}}\right) \leq 1.0$$

$R_{pg} = -8.68 \leq 1.0$

Maka $M_n = R_{pg} F_y S_{xc}$

$M_n = -8.68\,.\,345\,.\,17628894.8$

$M_n = 52.79\, x\, 10^{10}$Nmm = 52791.49 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:

- $F_{cr} = \left[F_y - (0{,}35 F_y)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right]$
- $\lambda = \frac{b_{fc}}{2t_{fc}} = \frac{300}{2.20} = 7.50$
- $\lambda_{pf} = \lambda_p = 0{,}38\sqrt{\frac{E}{F_y}} = 9.15$
- $\lambda_{rf} = \lambda_r = 0.95\sqrt{\frac{K_c E}{F_L}}$

$K_c = \frac{4}{\sqrt{h/t_w}} = 0.36$

$F_L = 0.7\,.\,F_y = 241.5$

$$\lambda_{rf} = \lambda_r = 0.95\sqrt{\frac{K_c E}{F_L}} = 16.40$$

So,

$F_{cr} = \left[F_y - (0{,}35 F_y)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] \quad = 321.44$

So that,

$M_n = R_{pg} F_{cr} S_{xc}$

$M_n = 49186365278.36\, Nmm$

$= 49186.37\ kNm$

4. Melt Wing Pull
   $M_n = F_y S_{xt}$
   $M_n = 345 . 17628894.8$
   $M_n = 6081968706\ Nmm$
   $M_n = 6081.97\ kNm$
   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.

Comparation 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:

Table 3.3 Comparative Results Table

No	*Code*	Nilai				Status
		M_u	Satuan	$ØM_n$	Satuan	
1	AASHTO LRFD *Bridge Design Spesifications* 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 ØMn 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 ØMn 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 <ØMn).

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.

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