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DESIGN UPPER STRUCTURE OF THE RIVER BAKI BRIDGE USES A STEEL FRAME

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ABSTRACT

In general, the bridge type is a reinforced concrete bridge, composite and steel frame. The Baki River Bridge was originally a 40 m long Wood type bridge connecting the village of Mengkopot - Tanjung Pisang because the condition of the Baki river bridge has been damaged especially on bridge floors and girders due to age, so it is planned to re-use the steel frame bridge structure. By using the steel bridge structure, it is expected that later it can be a comparison for the future bridge planning.

In this study, the structure of the planned bridge consists of bridge slabs, sidewalk slabs, backrest pipes, longitudinal and transverse girders, steel frames, wind ties, and joints. By following the SNI T-02-2005 bridge loading standard, SNI T-03-2005 for planning steel bridge structures and SNI T-12-2004 for planning concrete structures on bridges.

From the planning results obtained thickness 200 mm Bridge Slab and thick sidewalk slab 250 mm with a longitudinal girder profile used WF 600.200.11.17 mm steel with a distance between girders 1.75 m. Transverse girders of steel profiles WF 700,300.13.24 are used with the distance between girders 5 m. Used Steel profiles WF 400,400.13.21 as Main Frame and Steel WF profile 150.150.7.10 as profiles for wind ties.

‘Keywords: Steel frame, girder, connection,

1. Introduction

Bridges are one of the infrastructures needed for the sustainability of economic and social activities of an area or region. Economic and social activities can work well if the existing infrastructure conditions are also in good condition. So infrastructure conditions affect the quality of economic and social activities.

In essence, the condition of the bridge will certainly experience a decrease in strength, this is due to various factors such as lack of maintenance, the passing of the age limit of the plan, the burden that occurs exceeds the burden of the plan and many other factors. The same thing happened to the bridge that contacted the village of Mengkopot - Tanjung Pisang. This bridge with a span of 40 meters and a width of 5 meters is the main access (axis road) that connects between villages, sub-districts and even districts (Bengkalis - Meranti). This is feared to hamper activities and threaten the safety of the people passing on it. The condition of the bridge is currently damaged with decayed and perforated wood material. This is certainly very dangerous for the bridge user because it will cause accidents caused by the collapse of the bridge. Therefore it is necessary to take measures to restore the condition of the bridge so that it can serve its users well before the damage is made worse or may not be used again. So to overcome this, it is done on the bridge so that the condition of the bridge is as ideal as before.

The purpose of this study was to determine the dimensions of the Main Fence, Transverse Fence and Elongated Fence that are able to withstand the load acting on the bridge.

2. Basic Theory

2.1 The spread of load "D" in the transverse direction of the bridge

The "D" load must be arranged in a transverse direction such that it creates a maximum moment. Preparation BTR and BGT components of the cross-direction load must be the same.

a. Load factor "D"

Load factor "D" with a period of transient (temporary) can be seen in the following table. Load factor due to "D" lane load

Table 1. Load factors due to "D" lane loads

Jangka Waktu	Faktor Beban	
	$K_{S;TD;}$	$K_{U;TD;}$
Transien	1,0	1,8

(Source: SNI T-02-2005 article 6.3)

b. "T" Truck Loading

The loading of the "T" truck consists of semi-trailer truck vehicles which have the structure and weight as shown in Figure 3 below. The weight of each axle is spread to 2 equal equal loads which are the contact area between the wheel and the floor surface. The distance between the 2 axes can be changed between 4.0 m to 9.0 m to get the greatest influence on the direction of the bridge extending.

2.2 Position and spread of loading the truck "T" in the transverse direction of the bridge

Regardless of the length of the bridge or span arrangement, there is only one "T" truck that can be placed in a single lane of planned traffic. This "T" truck vehicle must be placed in the middle of the planned traffic lane as shown in the picture

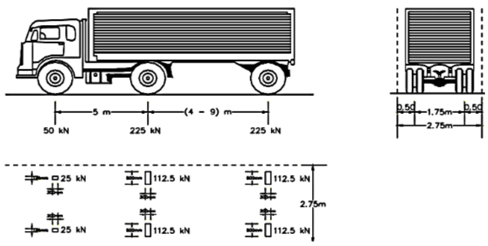

Figure 1. Truck Load on the bridge
(source: SNI T-02-2005)

For loading trucks "T" taken 30%

Table 2. Load factors for truck loads

Jangka Waktu	Faktor Beban	
	$K_{S;TT;}$	$K_{U;TT;}$
Transien	1,0	1,8

(*Source: SNI T-02-2005 article 6.4.1)*

FBD prices calculated are used in all parts of the lower building and the foundation is below the surface line, the FBD price must be taken as a linear circulation from the price on the land surface line to zero at a depth of 2 meters. For buried buildings, such as culverts and steel soil structures, FBD prices should not be taken less than 40% for zero depths and not less than 10% for a depth of 2 m. for the selected depth must be applied to the entire building.

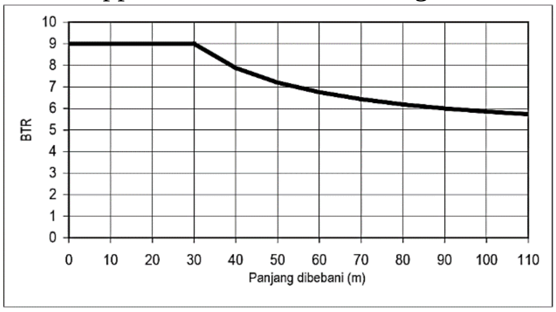

Figure 2. Load "D": BTR vs. length which is burdened
(Source: SNI T-02-2005)

- **Brake Style**

The operation of the forces directed at extending the bridge, due to brake force and traction, must be reviewed for both traffic directions. This influence is calculated as the braking force of 5% of the traffic department. This influence is calculated as the braking force of 5% of the "D" line load which is considered to be in all traffic lanes without multiplying by dynamic load factor and in one direction. The brake force is considered to work horizontally in the direction of the bridge axis with a 1.8 m high capture point above the floor surface of the vehicle. Lane load "D" here should not be reduced if the span length exceeds 30 m, the formula "D" **q = 9 kPa** is used.

3. Methodology

The steps in completing this research can be seen in Figure 3.

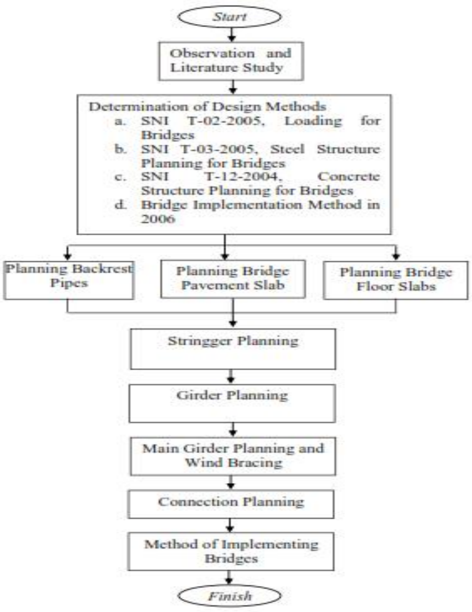

Figure 3. Flow Chart
(Source: Planning Data)

4. Results and Discussion

Construction Data of Sungai Baki bridge

a. The total length of the bridge = 40 m
b. The total width of the bridge = 9 m
c. Vehicle floor width = 7 m (2 x 3.5 m)
d. Sidewalk width = 2 x 1 m
e. Bridge Type = Steel Frame (Waren Truss Type)
f. Bridge frame height = 6.37 meters
g. The distance between girders transversely = 5 m
h. Concrete density = 2.5 t / m^3
i. Asphalt density = 2.2 t / m^3
j. Specific gravity of water = 0.98 t / m^3
k. Concrete floor thickness = 20 cm
l. Asphalt thickness = 10 cm

4.1 Planning Backrest Pipes

Workload:

Backrest for pedestrians must be planned for the imposition of service plan, ie q = w = 0.75 kN / m, with a height of 90 cm from the sidewalk surface.

Total Moment = 1192639.2 Nmm
Zx = 15971,637 mm2
Mn = Zx.Fy = 3833192.85 Nmm
ØMn = 0.9 x Mn = 3449873,57 Nmm
M Total <ØMn ... OK

So the profile used for the backrest is 2 (two) pipe profiles ∅ 60.5 mm with a thickness of 3.2 mm.

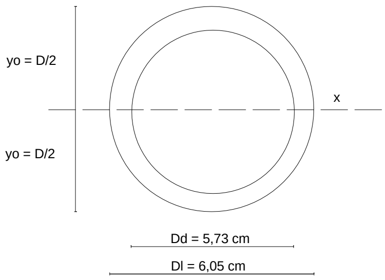

Figure 4. Dimension of Backrest Pipe
(Source: Planning Data)

a. Calculation of sidewalks
Moment calculation

Workload:
Moments due to weight alone
MMS = 5,625 kNm
Moments due to the burden of pedestrian life
MTP = 3.025 kNm
Moments due to additional life load
MMA = 0.147 kNm
Ultimate Moment
Mu = 13,657 kNm

Reinforcement

a. Concrete Quality fc '= 25 MPa
b. Steel Quality fy = 320 MPa
c. Thickness of concrete slab h = 200 mm
d. The distance of reinforcement to the outer side of concrete d '= 30 mm
e. Steel elastic modulus = 200000 MPa
f. Your ultimate plan moment = 13,656 kNm
g. The nominal moment of plan Mn = Mu / ϕ = 17.07 kNm
h. The diameter of reinforcement used is D - 16 mm
i. The required reinforcement distance, s = π / 4 x D2 x b / As = 593.4 mm2
j. Used reinforcement D 16 - 400
k. As' = π / 4 x D2 x b / s = 502.6 mm2
l. Longitudinal reinforcement is taken 50% of the main reinforcement
m. As' = 50% x As = 251.3 mm2
n. The diameter of reinforcement used is D - 13 mm
o. The required reinforcement distance, s = π / 4 x D2 x b / As = 528.12 mm
p. Used reinforcement D 13 - 400
q. As' = π / 4 x D2 x b / s = 331.8 mm2

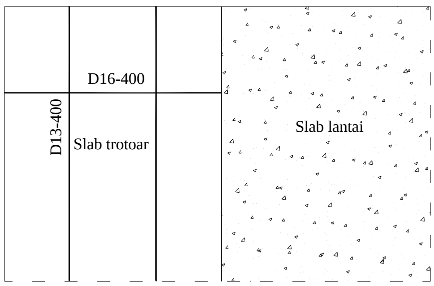

Figure 5. Reinforcement of Sidewalk Slabs
(Source: Planning Data)

b. Calculation of vehicle floor slabs

Moment Calculation

Workload:

The maximum moment in the slab is calculated based on the one way method with the following load:

QMS = 5 kN / m
QMA = 2,494 kN / m
PTT = 146.25 kN
PEW = 1.008 kN
ΔT = 12.5 ° C
K = moemen coefficient
S = 1.75 m

For evenly distributed loads (Q) M = k. Q. s^2
For centralized loads (P) M = k. p
For temperature loads (ΔT) = k T. Ec. s^3

Moments Due to Self Weight MS

Moment of support, MMS = 0.0833. QMS .s^2 = 1,275 kNm
Field moment, MMS = 0.0417. QMS. s^2 = 0.638 kNm

Moments due to additional MA dead load

Moment of support, MMS = 0.1041. QMA .s^2 = 0.795 kNm
Field moment, MMS = 0.0540. QMA. s^2 = 0.412 kNm

Moments due to TT truck loads

Moment of support, MMS = 0.1562. PTT .s = 39,97 kNm
Field moment, MMS = 0.1407. PTT. s = 36.01 kNm

Moments due to EW wind load

Moment of support, MMS = 0.1562. PEW. s = 0.275 kNm
Field moment, MMS = 0.1407. PEW. s = 0.248 kNm

Moment due to ET temperature

Moment of support, MMS = 5.62.10-7. α. . Ec. s3 = 0.008 kNm
Moment of the field, MMS = 2.8.10-6. α. . Ec. s^3 = 0.044

Table 3. Moments on Slab

No	Jenis Beban	Faktor Beban	Daya Layan	Keadaan ultimit	$M_{tumpuan}$ kNm	$M_{lapangan}$ kNm
1	Berat sendiri	K_{MS}	1.0	1.3	1.275	0,638
2	Beban mati tambahan	K_{MA}	1.0	2.0	0.795	0.412
3	Beban Truk	K_{TT}	1.0	1.8	39,97	36,01
4	Beban angina	K_{EW}	1.0	1.2	0,275	0,248
5	Pengaruh temperatur	K_{ET}	1.0	1.2	0,008	0,044

(Source: Data Processing)

Bridge Floor Slab Reinforcement

The reinforcement of the bridge floor plate is based on the result of combination 1

Table 4. Combination-1

No	Jenis Beban	Faktor Beban	$M_{tumpuan}$ kNm	$M_{lapangan}$ kNm	$M_{tumpuan}$ kNm	$M_{lapangan}$ kNm
1	Berat sendiri	1.3	1.275	0,638	1,65	0,83
2	Beban mati tambahan	2	0.795	0.412	1,59	0,82
3	Beban Truk	2	39,97	36,01	79,95	72,02
4	Beban angina	1	0,275	0,248	0.275	0,248
5	Pengaruh temperatur	1	0,008	0,044	0,0088	0,044
Total Momen Ultimit Slab			M_U		**83,48**	**73,96**

(Source: Data Processing)

Negative bending reinforcement

The steps to calculate the negative bending in the slab are described as follows;

a. Moment of a plan of support Mu = 83.48 kNm
b. Concrete quality: fc = 25 Mpa
c. Steel quality: BJ U39, fy = 390 Mpa
d. Thick concrete slab, h = 200 mm
e. The distance of reinforcement to the outer side of the concrete d '= 40 mm
f. Modulus of elasticity of steel, Es = 200000 Mpa

g. the form factor of the stress distribution of concrete ß1 = 0.85
h. Moment of a plan of support
i. Mu = 83.48 kNm
j. The nominal moment of plan Mn = Mu / φ = 104.35 kN / m
k. The diameter of reinforcement used = D 16
l. The required reinforcement distance s = 107.3 mm
m. Used D16-100 reinforcement
n. As = 2010,619 mm2
o. Reinforcement / shrinkage is taken 50% of the main reinforcement
p. As` = 50%. As = 1005,309 mm2
q. The diameter of reinforcement used D 13
r. The required reinforcement distance s = 132,03 mm
s. Used reinforcement D13-100 As' = 1327.32 mm2

Positive Bending Reinforcement

The steps for calculating positive bending in the slab are described as follows:

a. Moment of your support plan = 73.96 kNm
b. Concrete quality: fc = 25 Mpa
c. Steel quality: BJ U39, fy = 390 Mpa
d. Thick concrete slab, h = 200 mm
e. The distance of reinforcement to the outer side of the concrete d '= 40 mm
f. Steel elasticity modulus, Es = 200000 MPa
g. Moment of your support plan = 73.96 kNm
h. The nominal moment of plan Mn = Mu / φ = 92.46 kN / m
i. The diameter of reinforcement used D 16
j. The required reinforcement distance s = 122.96 mm
k. Used D16-100 reinforcement As = 2010,61 mm^2
l. Reinforcement for / shrinkage is taken 50% of the main reinforcement As` = 50%. As = 1005.30 mm^2
m. The diameter of reinforcement used D13
n. The required reinforcement distance s = 132.03 mm
o. D13-100 reinforcement is used As = 1327.32 mm^2

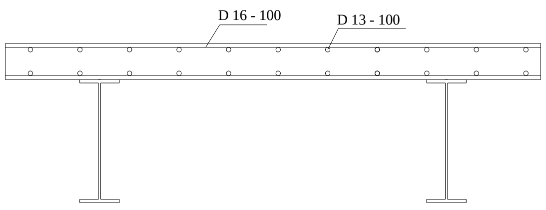

Figure 6. Reinforcement of vehicle slabs
(Source: Planning Data)

Slab Deflection Control

a. Concrete quality: fc = 25 MPa
b. The quality of BJ U39 steel, fy = 390 MPa
c. Steel elasticity modulus, Es = 200,000 MPa
d. Modulus of elasticity of concrete Ec = 0.043. = 23500 Mpa
e. Thick concrete slab, h = 200 mm
f. The distance of reinforcement to the outer side of the concrete d '= 40 mm
g. The area of slab reinforcement As = 2010,619 mm^2
h. Slab span length, Lx = 1750 mm
i. Wide slab review, b = 1000 mm
j. Centralized Load, PTT = 146.25 kN
k. Equivalent Load, Q = QMS + QMA = 7,494 kN / m
l. The total deflection that occurs δ tot <Lx / 240 = 7.291 mm
m. Gross inertia of slab cross section (Ig) = 1/12. b. h^3 = 7 x 108 mm^3
n. Modular flexural failure modulus fr = 0.7. = 3.5 MPa
o. Comparison value of elasticity modulus n = Es / Ec = 8.5
p. n. As = 17111.65 mm2
q. neutral line distance to the upper side of the concrete
r. c = n. As / b = 17.11 mm
s. crack section inertia transformed :
   Icr = 1/3. b. c3 + As. (d - c) 2 = 3.51x108 mm4
t. Yt = h / 2 = 100 mm
u. Moment of cracking Mcr = fr. Ig / yt = 2,33x107 mm4
v. Maximum moment due to load (without load factor)
   Ma = 1/8. Q. Lx2 + ¼. P. Lx = 66.85 kNm
   Ma = 6.69 x107 Nmm
w. Inertia is effective for calculating deflection
   Ie = (Mcr / Ma) 3. Ig + (1-Mcr / Ma) 3) .Icr = 3.64 x 108 mm4
x. Instant elastic deflection due to dead load and live load:
   Q = 7,494 N / m
   P = 146250 N
   e = 2.013 mm
y. Ratio of bridge floor slab reinforcement
z. ρ = As / b .d = 0,0125
aa. Time dependency factor for dead load (period> 5 years), value ξ = 2.0
bb. Long-term deflection due to creep and shrinkage:
   δg = 0.131
cc. The total deflection on the bridge floor plate: Lx / 240 = 7,291
   ottot <δe + δg = 2.144
   2,144 <7,291 ... .OK

c. Calculation of the Fence Stringger

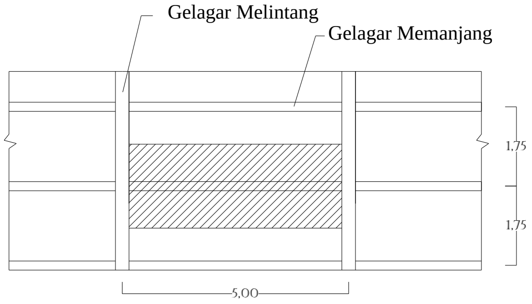

Figure 7. Distance of Stringger and Girders
(Source: Planning Data)

The distance between Stringger = 1.75 m
The distance between girder = 5 m
Steel elasticity modulus (E) = 200000 MPa
Melting stress (fy) = 410 MPa
Ultimit voltage (fu) = 550 MPa

Table 5. Profile Data WF.600.200.11.17

G	106 kg/m	Zx	2590 cm^3
Ix	77600 cm^4	Zy	228 cm^3
Iy	2280 cm^4	H	600 mm
A	134,4 cm2	t f	17 mm
Ix	24,03 cm	B	200 mm
Iy	4.12 cm	t w	11 mm

(Source: PT. Gunung Garuda)

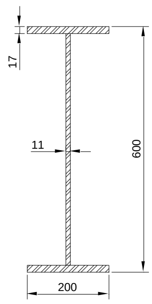

Figure 8. Profile of WF 600.200.11.17
(Source: Planning Data)

Workload:

a. Life Expenses Due to the "D" load (load factor = 1.8)
b. The load is evenly distributed (BTR)
c. L = 40 m> 30 m
d. q = 9.0 x (0.5 +) = 9.0 x = 7.875 Kpa = 787.5 kg / m^2
e. The distance between the girders extends 1.75 m.
f. qL = 787.5 x 1.75 x 1.8 = 2480.625 kg / m = 24.81 kN / m

Line Load
P = 49.0 KN / m = 4900 kg / m
L = 40 m then FBD = 40%
P '= 4900 x 1.75 x (1 + 40%) x 1.8 = 21609 kg

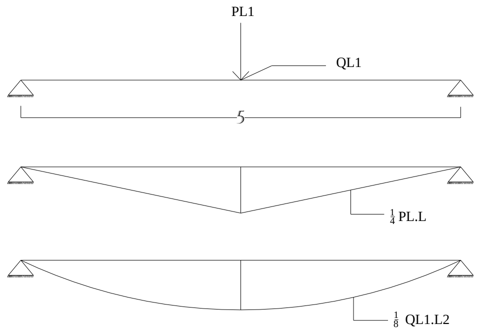

Figure 9. Loading due to BTR and Line Loads

ML_1 $= (\frac{1}{8} q_L \cdot L^2) + (\frac{1}{4} P_1 \cdot L)$
$= (\frac{1}{8} x\ 2480{,}625\ x\ 5^2) + (\frac{1}{4}\ 21609)$
= 34763,2 kg.m

Moments due to truck load "T"
According to SNI T-02-2005, it is known:
T = 112.5 kN / m

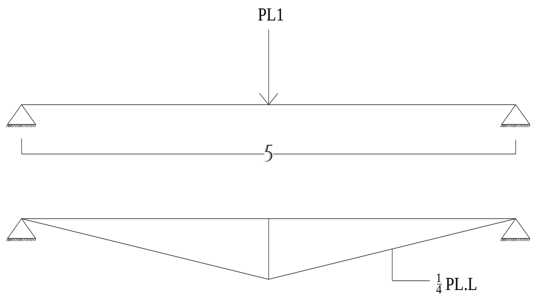

Figure 10. Loading due to truck load
(Source: Planning Data)

ML_2 = ½ x PTT x L x K_{UTT}
= 146,25 x 5 x 1,8
= 658,13 kN.m = 65813 kg.m

Because ML1 <ML2, the moment is used due to the "T" Truck load that is ML2 = 65813 kg.m

Girder

For the initial cross-beam planning, the WF profile is selected with the profile dimensions of WF. 700.300. 13.24

(Source: PT. Gunung Garuda)

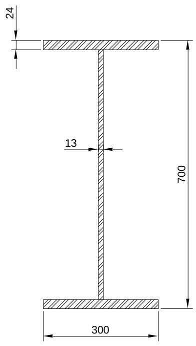

Figure 11. Steel profile 700,300.13.24
(Source: Planning Data)

Workload:

Dead load

Asphalt layer weight = 2200 Kg / m
Curb weight:
Own weight of concrete plate = 4063 Kg m
Steel deck weight = 65.33 Kg / m
Sidewalk = 4128 kg / m
Qd = 6327.82Kg/m
Σ MB = 0
Ra = 11827.82 kg
MD = (11827.82 x 4.5) - (4128 x 1 x 4) - (2200 x 1.75 x 3.5) = 23239 kg.m

Due to "D" Load

Flat Divided Load (BTR)

L = 40 m> 30 m

$$q = 9{,}0 \ x \ (0{,}5 + \frac{15}{L}) = 9{,}0 \ x \ \left(0{,}5 + \frac{15}{40}\right) =$$

7,875 Kpa = 787,5 kg/m^2
Then q ' = 5 x 787.5 x 1.8 = 7088 kg/ m^2
100% Load → q '= 7088 kg / m
50% load → q '= 3543.75 kg / m

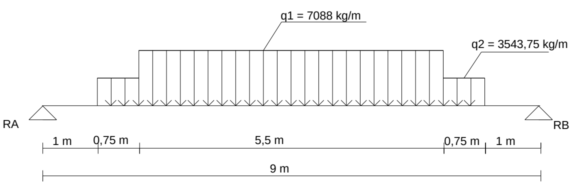

Figure 12. Distribution due to "D" load
(Source: Planning Data)

Σ MB = 0

$$Va = \frac{(q2 \ x \ 0{,}75) + (q1 \ x \ 5{,}5) + (q2 \ x \ 0{,}75)}{2}$$

$$= \frac{(3544 \ x \ 0{,}75) + (\ 7088 \ x \ 5{,}5) + (3544 \ x \ 0{,}75)}{2}$$

= 22148 kg

Mmax = (Va x 4,5) – (q2 x (2,75 + 0,375)) – (q1 x 1,375) = 88594 kg.m

G	185 kg/m	Zx	5760 cm^3
Ix	201000 cm^4	Zy	722 cm^3
Iy	10800 cm^4	H	700 mm
A	235,5 cm2	t f	24 mm
Ix	29,3 cm	B	300 mm
Iy	6,78 cm	t w	13 mm

Table 6. WF.700.300.13.24 Profile Data

Line load

P = 49 kN / m = 4900 kg / m
L = 40 m then DLA (Dynamic Load Alloance) = 40%
Where the distance between girders transverses = 5.00 m.
P '= (1 + 0.4) x 4900 x 1.8 = 12348 kg
100% Load → P1 '= 12348 x 5.5 x 100% = 67914 kg / m
50% Load → P2 '= 12348 x 0.75 x 50% = 4630.5 kg / m

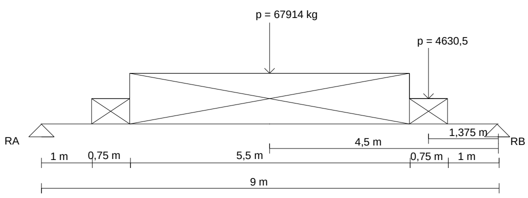

Figure 13. Distribution due to "P" load
(Source: Planning Data)

ΣMB = 0

R_A . 9 – (67914 x 4,5) – (4630,5 x 1,375) = 0

$R_A = \frac{311980}{9} =$ 34664 kg

M_{L2} = (Ra x 4,5) – (P_1’ x 0)
= (34664x 4,5) – (67914 x 0)
= 155989,96 kg.m

The total moment due to the "D" load is:

M_{Total} = M_{L1} + M_{L2} = 88593,8 + 155989,96
= 244583,718 kg.m

"T" Truck Load

P = (1 + 0,4) x [(2500 x (1/5)) + 11250 + (11250 x (1/5))] x 1,8 = 35280 kg

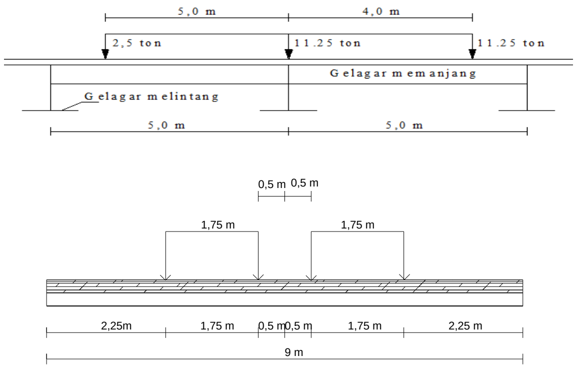

Figure 14. Distribution of "T" Load

(Source: Planning Data)

ΣMB = 0

V_A . 9 – (35280 x 6,75) – (35280 x 5) – (35280 x 4) – (35280 x 2,25) = 0

$V_A = \frac{635040}{9} = 70560$ kg

Maximum moment in the middle of the span:

Mmax = R_A x 4,5 – (P x 0,5) – (P x 1,25)
= (70560 x 4,5) – (35280 x 0,5) – (35280 x 1,25) = 255780 kg.m

The biggest moment is used due to the load "T" of 255780 kg.m

Main Girder

Load Dead plate for Stringger

Workload:

a. For edge Girder

Sidewalk = 0,25 x 1 x 25 = 6,25 kN/m

Slab = 0,2 x ((1,75 / 2) + 1) x 25
= 9,375 kN/m

Total = 6,25 + 9,375 = 15,63 kN/m

So for the edge girder, the Load Dead Plate is 15.63 kN / m

b. For central Gathering

Slab = 0,2 x 1,75 x 25 = 8,75 kN/m

So for the girder it is obtained the Dead Load plate of 8.75 kN / m

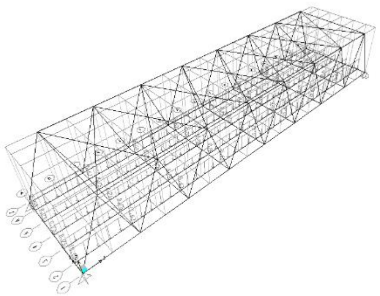

Figure 15. Input Dead Load Plate
(Source: Planning Data)

Additional Dead Load for Stringger

Workload:

a. For edge Girder

Asphalt = 0,1x(1,75/2)x22 = 1,925 kN/m

Rainwater = 0,03 x ((1,75 / 2) + 1) x 9,8
= 0,551 kN/m

Bondek = 0,101 x ((1,75 / 2) + 1) \
= 0,188 kN/m

Total = 1,925 + 0,551 + 0,188
= 2,665 kN/m

So for edge girders, the Additional Dead Load is 2,665 kN / m

b. For Central Gathering

Asphalt = 0,1 x 1,75 x 22 = 3,85 kN/m

Rainwater =0,03x 1,75 x 9,8 = 0,515 kN/m

Bondek = 0,101 x 1,75 = 0,176 kN/m

Total = 3,85 + 0,515 + 0,176
= 4,54 kN/m

So for the center girder, the Additional Dead Load is 4.54 kN / m

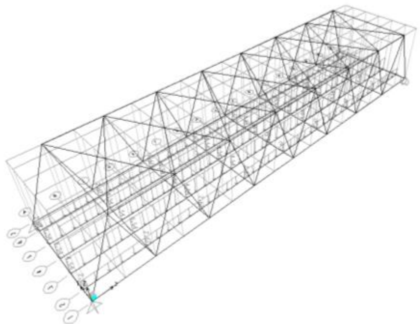

Figure 16. Additional Dead Load Input
(Source: Planning Data)

Life Expenses

Workload:

a. The burden is evenly distributed

L = 40 m ⟶ $q = 9\ x\ \left(0{,}5 + \frac{15}{L}\right)$

$q = 9\ x\ (0{,}5 + \frac{15}{40})$

= 7,875 kPa

= 7,875 kN/m^2

For edge girder

BTR = (1,75 / 2) x 7,875
= 6,891 kN/m

So for the edge girder the BTR is 6.891 kN / m

For Central Gethering

BTR = 1,75 x 7,875 = 13,78 kN/m

So for the girder the BTR is obtained at 13.78 kN / m

a. Line Load

P = 49 kN/m

FBD = 0,4

Q BGT = 2 x 49 x 2,75 = 269,5 kN

BGT 100%= 1,4 x 269,5 = 377,3 kN/m

BGT 50% = 50% x 269,5 = 188,7 kN/m

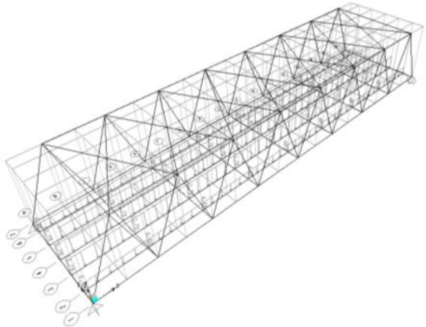

Figure 17. BTR input
(Source: Planning Data)

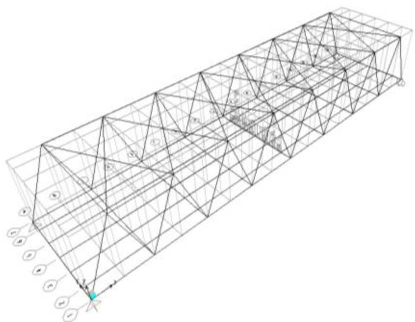

Figure 18. BGT input
(Source: Planning Data)

Load T
Workload:

S = 1,75 / 3,4 = 0,515
TT_1 = 25 x (1 + 30%) = 32,5
TT_2 = 112,25 x (1 + 30%) = 146,25

a. For edge girder
TT_1 = 0,515 x 32,5 = 16,728 kN
$TT2_1$ = 0,515 x 146,25 = 75,276 kN

b. For Central Gethring
TT_1 = 0,515 x 32,5 x 2 = 33,46 kN
$TT2_1$ = 0,515 x 146,25 x 2 = 150,6 kN

Wind Load
Workload:
Vihicle Height = 2 m
Review of each = 1 m

Ab $= \frac{(35+40)}{2} \times 6,37 \times 30\%$
= 71,66 m^2

T_{EW1} = 0,0006 x 1,2 x 35^2 x 71,66 = 63,21 kN

a. Number of joint
Bottom = 9
Top = 8
Total = 17

b. T_{ew1} per joint = T_{ew} / 17
= 63,21 / 17 = 3,718

T_{EW2} = 0,0012 x Cw x Vw^2 x Ab
= 0,0012 x 1,2 x 35^2 x 2
= 3,528 kN

T_{EW2} per joint = 3,528 x 40

= 141,1 kN/m / 9 = 15,68 kN/m

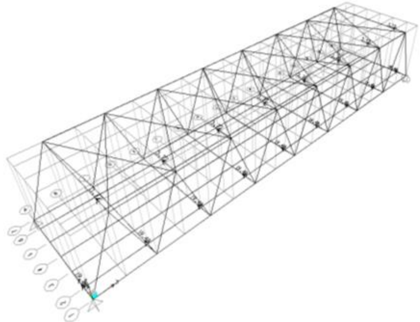

Figure 19. Input of wind load
(Source: Planning Data)

Pedestrian Expense
Workload:

Curb area = 40 x 1
= 40 m^2
Getting on from the graph = 4 kPa

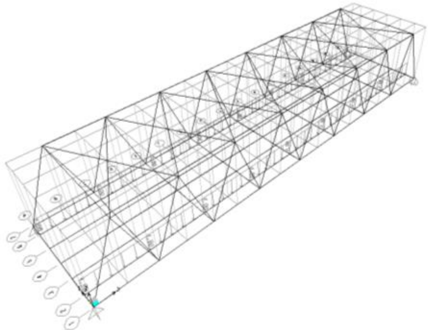

Figure 20. Pedestrian Input
(Source: Planning Data)

Brake Style
Workload:

q = 9 kPa
TB = 0,05 x ((q x L) + 49) x 7 = 143,2 kN

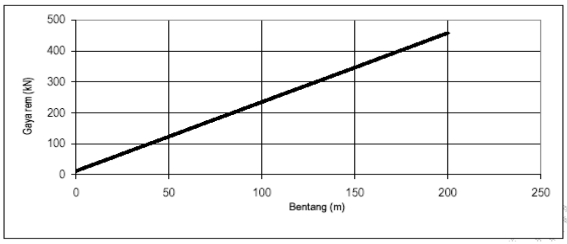

Figure 21. Brake force per lane 2.75 m (KBU)
(source: SNI T-02-200)

From the graph is obtained 100 kN
The maximum is 143.2 kN

a. Number of Joints
Girder = 9
Stringger = 5
Total joint = 45
Intersection load = 143,2 / 45
= 3,181

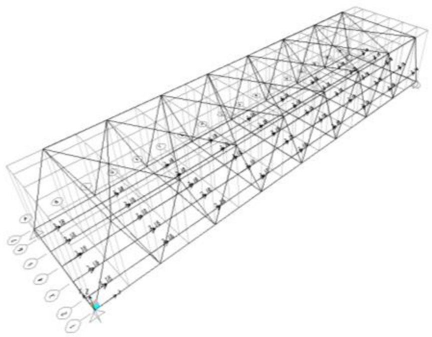

Figure 22. Brake Style Input
(Source: Planning Data)

Planning Dimensions of Main Main Profile

Main girder Is the main part of building construction above, which functions to forward all loads received by the upper building and forwarded to the lower building.

From the output of SAP 2000, the largest compressive axial force Pu = 538729.1 kg is obtained

Stem dimensions are tried using the WF.400.400.13.21 profile

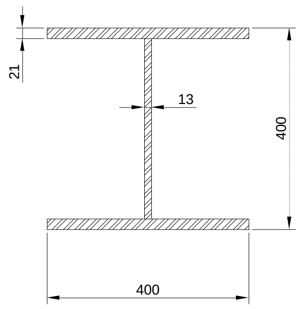

Figure 23. Steel Profile WF.400.400.13.21
(Source: Planning Data)

Table 7. Steel profile data WF.400.400.13.21

A	218,7 cm^2	iy	10,1 cm
Ix	66600 cm^4	Zx	3330 cm^3
Iy	22400 cm^4	Zy	1120 cm^3
ix	17,5 cm		

(Source: PT. Gunung Garuda)

Calculating the Radiation of Giration (r)

$$rx = \sqrt{\frac{I_x}{A_g}} = \sqrt{\frac{66600}{218,7}} = 17,45 \text{ cm}$$

$$ry = \sqrt{\frac{I_{y_y}}{A_g}} = \sqrt{\frac{22400}{218,7}} = 10,12 \text{ cm}$$

Lean parameters (λc)

$$\lambda_c = \frac{K.L}{r} . \sqrt{\frac{F_y}{\pi^2 E}}$$

Where :

K = Factor of effective length = 0,7

$$\lambda c = \frac{0,7\,x\,500}{10,12} \sqrt{\frac{4100}{3,14^2\,x\,(2,0\,x 10^6)}} = 0,498 \text{ cm}$$

Calculate Critical cross section voltage (Fcr)

$\lambda c \leq 1,5 \Rightarrow$ Fcr = ($0,658^{\lambda c^2}$). Fy

Fcr = ($0,658^{(0,498)^2}$) x 4100

= 3695,1075 kg/cm2

Pn = Fcr . Ag

Then : $\phi_c . P_n \geq P_u$

0,85 x 3695,107 x 218,7 ≥ 538729,13 kg

686902 kg > 538729,13 kg (safe profile)

Planning Diagonal Rod Dimensions

Planning Diagonal Rod Dimensions Press (Compression)

From the results of SAP 2000 output, the largest compressive axial force Pu = 275632.32 kg is obtained

Stem dimensions are tried using the WF.400.400.13.21 profile

Table 8. Profile of WF.400.400.13.21

A	218,7 cm^2	iy	10,1 cm
Ix	66600 cm^4	Zx	3330 cm^3
Iy	22400 cm^4	Zy	1120 cm^3
ix	17,5 cm		

(Source: PT. Gunung Garuda)

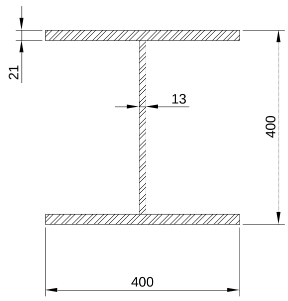

Figure 24. Profile of WF.400.400.13.21
(Source: Planning Data)

Calculating the Radiation of Giration (r)

$$rx = \sqrt{\frac{I_x}{A_g}} = \sqrt{\frac{66600}{218,7}} = 17,45 \text{ cm}$$

$$ry = \sqrt{\frac{I_{y_y}}{A_g}} = \sqrt{\frac{22400}{218,7}} = 10,12 \text{ cm}$$

Lean parameters (λc)

$$\lambda_c = \frac{K.L}{r} . \sqrt{\frac{F_y}{\pi^2 E}}$$

Where :

K = Factor of effective length = 0,7

$$\lambda c = \frac{0,7\,x\,500}{10,12} \sqrt{\frac{4100}{3,14^2\,x\,(2,0\,x 10^6)}} = 0,498 \text{ cm}$$

Calculate Critical cross section voltage (Fcr)

$\lambda c \leq 1,5 \Rightarrow F_{cr} = (0,658^{\lambda c^2})$. Fy

F_{cr} = ($0,658^{(0,498)^2}$) x 4100 = 3695,107 kg/cm^2

$P_n = F_{cr} . Ag$

Then : $\phi_c.P_n \geq P_u$

0,85 x 3695,107 x 218,7 $\geq$ 275632,32 kg

686902 kg > 275632,32 kg *(safe profile)*

Planning the Wind Bond Dimension

a. Planning of compressive wind bond dimensions (Compression)

From the output of SAP 2000, the largest compressive axial force Pu = 5328.78 kg

Stem dimensions are tried using the WF.150.150.7.10 profile

Table 9. Profile of WF.150.150.7.10

A	40,14 cm^2	iy	3,75 cm
Ix	1640 cm^4	Zx	218 cm^3
Iy	563 cm^4	Zy	75,1 cm^3
ix	6,39 cm		

(Source: PT. Gunung Garuda)

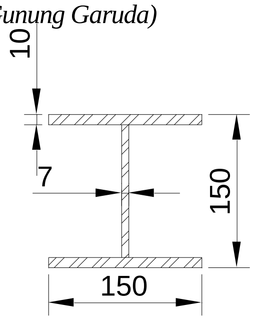

Figure 25. Steel Profile WF.150.150.7.10
(Source: Planning Data)

Calculating the Radiation of Giration (r)

$$rx = \sqrt{\frac{I_x}{A_g}} = \sqrt{\frac{1640}{41,14}} = 6,3919 \text{ cm}$$

$$ry = \sqrt{\frac{I_{y_y}}{A_g}} = \sqrt{\frac{563}{40,14}} = 3,7451 \text{ cm}$$

Lean parameters (λc)

$$\lambda_c = \frac{K.L}{r} . \sqrt{\frac{F_y}{\pi^2 E}}$$

Where :

K = Factor of effective length = 0,7

$$\lambda c = \frac{0,7 \, x \, 500}{3,74} \sqrt{\frac{4100}{3,14^2 \, x (2 \, x \, 10^6)}} = 1,34 \text{ cm}$$

Calculate Critical cross section voltage (Fcr)

$\lambda c \leq 1,5 \Rightarrow F_{cr} = (0,658^{\lambda c^2}).\ Fy$

$F_{cr} = (0,658^{(1,34)^2})x\ 4100 = 1918,8\ kg/cm^2$

$P_n = F_{cr}\ .\ Ag$

Then : $\phi_c.P_n \geq P_u$

0,85 x 1918,8 x 40,14 $\geq$ 5328,78 kg

65467,43 kg > 5328,78 kg *(safe profile)*

b. Planning Dimensions of Diagonal Rods (Tension)

From the results of SAP 2000 output obtained the largest axial tensile force Pu = 4939.2 kg

Stem dimensions are tried using the WF.150.150.7.10 profile

Table 10. WF.150.150.7.10 profile

A	40,14 cm^2	iy	3,75 cm
Ix	1640 cm^4	Zx	218 cm^3
Iy	563 cm^4	Zy	75,1 cm^3
ix	6,39 cm		

(Source: PT. Gunung Garuda)

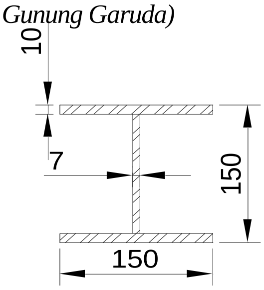

Figure 26. Steel Profile WF.150.150.7.10
(Source: Planning Data)

Check profile ratio:

Because the two elements (flanges) of the cross section are connected while the body elements are not connected, the profile is checked using the equation:

$$\frac{bf}{d} \geq 0,67$$

Where : bf = 40

d = 40

$$\frac{40}{40} \geq 0,67$$

1 > 0,67

The calculation of press rod dimensions includes:

Calculate nominal area

A325 bolts in diameter are used = ft = 2.22 cm

Bolt hole width = 1 inch = 2.54 cm

Nominal area of the plate:

A¬n = 40.14 - (2.54 x 1) = 37.6 cm^2

Plate net area (effective area of cross section) based on:

Ac = U. An

Where :

U = the reduction coefficient whose value no more than 85%

Then: Ac = U. An = 0.85 x 37.6 = 31.96 cm^2

Strength control design

Based on the gross cross section:

Where :

= Resistance factor for melting limit state (0.90)

Mr = nominal strength of pull rod (kg)

Fy = steel melting voltage = 4100 kg / cm^2
Ag = gross area cross section = 218.7 cm^2
So: $\varphi_t T_n = \varphi_t F_y A_g \geq P_u$
0.90 x 4100 x 40.14 ≥ 4939.2 kg
148117 kg ≥ 4939.2 kg (safe profile)
Based on a clean cross section
Where :
ϕt = Resistance factor for melting limit state (0.75)
Mr = nominal strength of pull rod (kg)
Fu = tensile strength of steel structure = 4900 kg / cm^2
Ac = Effective net area between pull rod = 181.36 cm^2
so: $\phi_t T_n = \phi_t F_t A_c \geq Pu$
0.75 x 4900 x 181.36 ≥ 4939.2 kg
666498 kg > 4939.2 kg (safe profile) From the results of the two criteria above, a smaller design strength is taken, namely: 666498 kg ≥ Pu = 4939.2 kg

5. Connection Planning

5.1 Stringger Connections with Transverse Girders

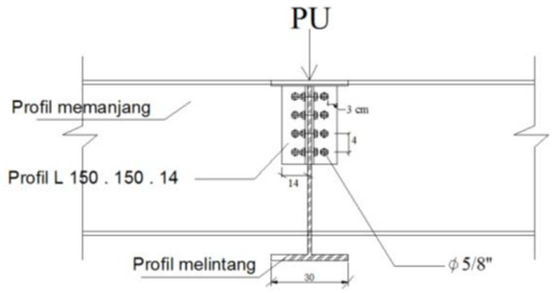

Figure 27. Stringer girder joint and transverse girder
(Source: Planning Data)

A bolt A 325 Ø 5/8 inch is used.
Ø Bolt = 5/8 inch = 1,588 cm
Area Ab = 1.981 cm^2
Ø bolt hole = $\left(\frac{5}{8}+\frac{1}{8}\right)$ = inch = 1,905 cm
Fub = Bolt tensile strength = 120 ksi = 8274 kg / cm^2 = 1 ksi = 68.95 kg / cm^2

Rod Joints in Main Fence

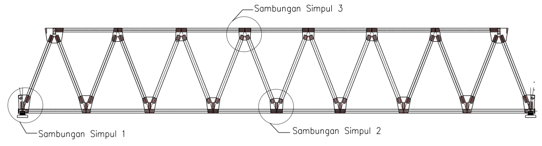

Figure 28. Stem Connections (Knot) in Main Fence
(Source: Planning Data)

Connections to the frame use A325 high quality bolts.

. **Table 11.** Data Bolts Used

Material strength (F_u^b) = 120 ksi	8274,000 kg/cm^2
Tensile strength	4654,125 kg/cm^2
Shear strength	2420,145 kg/cm^2
Bolt Diameter (∅ 7/8″)	22,2 mm
Bolt Hole Diameter	2,54 cm
Bolt Area (Ab)	3,869 cm^2

5.2 Rod Joints on the Wind Bracing

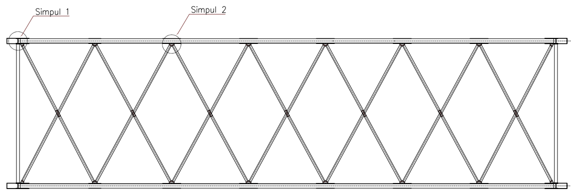

Figure 29. Wind Bond Connection (Knot)
(Source: Planning Data)

Connections to wind ties use high quality A325 bolts.

Table 12. Data Bolts Used

Material strength (F_u^b) = 120 ksi	8274,000 kg/cm^2
Tensile strength	4654,125 kg/cm^2
Shear strength	2420,145 kg/cm^2
Bolt Diameter (∅ 7/8″)	22,2 mm
Bolt Hole Diameter	2,54 cm
Bolt Area (Ab)	3,869 cm^2

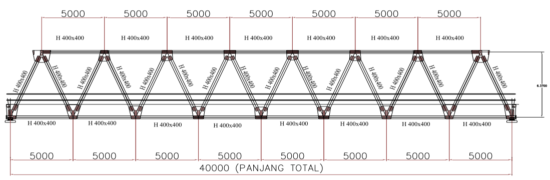

Figure 30. Side view
(Source: Planning Data)

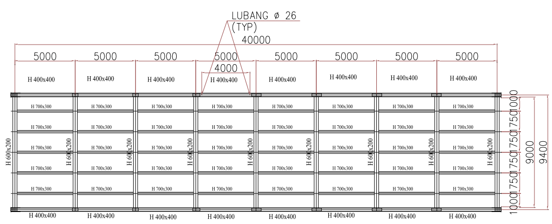

Figure 30. Side view
(Source: Planning Data)

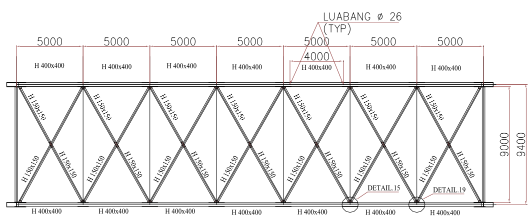

Figure 32. Floor Plan for Upper Wind
(Source: Planning Data)

6. Placement Planning

Ealstomers are planned based on vertical loads that work. From the results of calculations in SAP 2000 obtained the maximum vertical load on the bridge support of 366.75 tons. Use elastomer bearing production of PT. Main Ralico Rubber with dimensions of 350 x 700 mm with a maximum number of layers is 7 layers using a thickness of 12 mm per layer. This elastomeric dimension is able to withstand vertical loads so that it is 367.5 tons so that this dimension is safe for use in the bridge construction of this steel truss tray.

Figure 33. Elastomeric Bearings
(Source: http: //www.bridgebearing.0rg/bridgebearing/laminated-elastomeric-bearing-pad.html)

7. Conclusion

River Baki Bridge is designed for 40 meters. From the results of the analysis and calculation of bridge planning, there are some conclusions as follows.

a. Pipe Dimensions Backrest used is 60.5 mm for outside diameter and 57.30 for inner diameter.
b. The bridge floor plate thickness used is 200 mm.
c. The thickness of the sidewalk slab used is 250 mm.
d. The Transverse Fence Profile obtained from the calculation is a WF 700 x 300 x 13 x 24 profile.
e. The Stringer longated Fence Profile obtained from the calculation is the WF 600 x 200 x 11 x 17 profile.
f. The Main Fence Profile obtained from the calculation is the WF 400 x 400 x 13 x 21 profile.
g. Wind Bracing Profile obtained from the calculation is WF profile 150 x 150 x 7 x 10.

8. Bibliography

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