DESIGN OF COMPOSITE GIRDER STRUCTURE BRIDGE OF SELUANG-1 RIVER PT LIFERE AGRO KAPUAS, KAPUAS DISTRICT

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ABSTRACT

The bridge at the Seluang-1 river is located around the palm oil plantation land owned by PT Lifere Agro Kapuas, Kapuas Regency, Central Kalimantan. In this Seluang-1 river, a bridge is planned to be built to facilitate the mobilization of palm oil plantation crops and other matters as well as transportation in the PT Lifere Agro Kapuas area because before there was a bridge the transportation traffic was cut off by the river so it had to circle quite a long way. The bridge is designed as a bridge with composite girder structure type.

The methodology in the design of loading uses the SNI 1725-2016 concerning on the Loading Standards for Bridges and SNI 2833-2016 concerning on the Earthquake Resilience Planning Standards for Bridges. For methodology in designing concrete structures refer to SNI 2847-2013 concerning Structural Concrete Requirements for Buildings and methodology in designing steel structures refers to SNI 1729-2015 concerning Specifications for Structural Steel Buildings. The material used for abutment uses reinforced concrete material. The methodology in calculating the bearing capacity of the foundation uses the method by Mayerhof and also the method by Kazuto Nakazawa, while the methodology in calculating the lateral bearing capacity uses the Broms method, with the efficiency of the pile using a graph by O'Neill.

From the results of topographic measurements taken a bridge design with a span of 30 m with a total bridge width of 7 m. The slab design is 25 cm thick with the compressive strenght of concrete is fc' 30 MPa (K-350). The steel girder beam used WF Profile 1350.800.100.130 and the diaphragm beam used WF Profile 250.125.6.9 with BJ55 steel quality (fy 410 MPa). Whereas in the lower structure, the abutment designed with a height of 350 cm, a width of 320 cm and a length of 850 cm, was used with compressive strenght of concrete is fc' 30 MPa (K-350). In the foundation used Spun Pile type piles with a diameter of 60 cm with a depth of 30 m piling as much as 8 piles on one abutment. Obtained Q_{allow} = 116,37 tons > Q_{load} = 114,69 tons so that the foundation is declared safe. The planned budget for the construction of a bridge on the Seluang-1 river is Rp 8.990.566.000,00.-

Keywords: Bridge,   composite,   steel   girder,   abutment,   spun   pile

1   Introduction

1.1   Background

Today, infrastructure in the field of transportation is important for human life in undergoing its activities, including industry. Supporting infrastructure results in increased time efficiency in production. On the road in the palm oil plantation land owned by PT Lifere Agro Kapuas, Kapuas Regency, Central Kalimantan Province, it is felt that it is not efficient in transporting harvested products from oil palm plantations to harvest processing plants and transportation in the area due to having to circle the road far enough due to the interruption of the roads by the river. One of them is in a location named Seluang-1. At this Seluang-1 location, a bridge is planned to be built to facilitate the mobilization of the transportation of harvested oil palm plantations and other matters as well as transportation within the PT Lifere Agro Kapuas area.

1.2   Problem's Formulation

From the description above, the scope of the issues that will be discussed in this thesis are as follows:

1.   Design a safe and efficient bridge superstructure (in terms of cost) by taking into account the topography and class requirements of the bridge.

2.   Design a safe and efficient bridge substructure (in terms of cost) but still meet the technical requirements by taking into account the results of the soil investigation at the site.

3.   The design results are in the form of technical drawings.

4.   Cost Budget Plan.

1.3   Design Purpose

The purpose of writing this final project is as follows:

1.   Obtain bridge superstructures design such as sidewalks, vehicle floors, main girders and diaphragms.

2.   Getting the design of bridge substructures such as abutments, oprit, and bridge foundations.

3.   Getting the design results in the form of technical drawings.

4.   Calculating the Cost Budget Plan

1.4   Design Benefits

The benefit of this thesis is to get the superstructures and substructures design of the Seluang-1 bridge location at PT Lifere Agro Kapuas that meets technical requirements but remains efficient so that it can later be utilized to facilitate the mobilization of crops and so on.

2   Theory Basis

2.1   Bridge Design Criteria

In designing a bridge, the analysis carried out is as follows:

a.   Calculation of bridge loading.

b.   Calculation of superstructures of the bridge in the form of a vehicle floor, girder and diaphragm.

c.   Calculation of substructures of the bridge covering abutments, pillars and foundations.

2.2   Location Data

The design was carried out at PT Lifere Agro Kapuas, Seluang-1 location, Kapuas Regency, Central Kalimantan. With coordinates UTM 220081 Easting, 9700069 Northing with UTM 50 Zone and Hemishphere South. The data obtained are secondary data in the form of data from topographic measurements, as well as data from soil investigations. Map of design location can be seen in Figure 2.1.

Figure 2.1   Design Location Map

(Source: Google Earth)

2.3   Bridge Loading

The load on the bridge that was designed uses the SNI 1725-2016 concerning Loading for the Bridge and SNI 2833-2016 concerning Earthquake Resilience Planning Standards for the Bridge. The type of load is divided into two types namely fixed / permanent loads and transient loads.

2.4   Vehicle Floor Slab

The vehicle floor is in the form of reinforced concrete slabs and is designed with reference to SNI 2847-2013 Structural concrete requirements for buildings.

2.5   Bridge Main Girder

The main girder in the form of steel profiles along with other steel components such as bolt connections and shear connectors are designed using the LRFD method with reference to SNI 1729-2015 Specifications for Structural Steel Buildings.

2.6   Bridge Substructure

The substructuresof the bridge consists of the design of the bridge abutment and foundation. The load of abutments uses the SNI 1725-2016 reference, namely the Loading for Bridges and SNI 2833-2016 Earthquake Resilience Planning Standards for Bridges. For reinforcement of the abutment itself is designed using the reference SNI 2847-2013 structural concrete requirements for buildings.

Calculation of bearing capacity of the pile foundation using the method by Mayerhof and also the method by Kazuto Nakazawa. Then, for the calculation of the lateral bearing capacity of the pile foundation using the method by Broms. Meanwhile, for the efficiency of the pole using a graph by O'Neill.

2.7   Cost Budget Plan

From the design drawings, it can be calculated the volume of each bridge material which is grouped into several divisions according to the type of work in accordance with Bina Marga standards.

3   Design Methodology

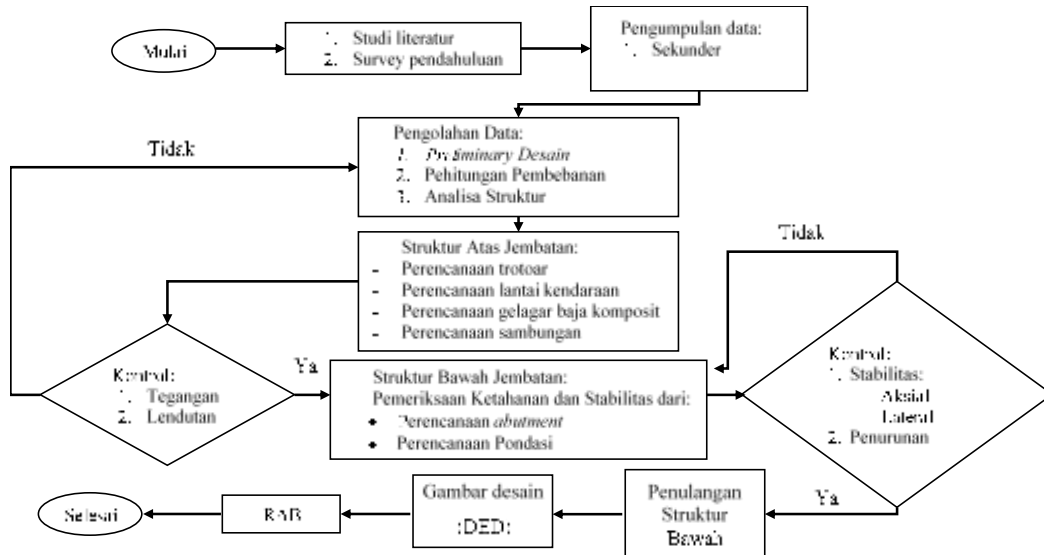The method in design is illustrated in the flow chart.

Figure 3.1   Design Flowchart

4   Design Result

4.1   Bridge Technical Data

From the secondary data obtained a bridge with a span length of 30 m and class bridge B (with a vehicle floor width of 6 meters and the sidewalk width on each side - 0.5 meters).

4.2   Calculation of Bridge Superstructures

4.2.1 Backrest Pipe Design

From the results of the design, the backrest pipe $\emptyset 76,3 \, mm$ - thickness = 2,4 mm is used and controls the material stress $(\sigma) \leq$ allowable stress (σ'):

$$(\sigma = 1.804,838 \, kg/cm^2) \leq (\sigma' = 3.666,667 \, kg/cm^2) \ldots \textbf{OK}$$

4.2.2 Backrest Beam Design

From the results of the design, the backrest beam WF 100.100.6.8 BJ 55 is used and controls the material stress $(\sigma) \leq$ allowable stress (σ'):

$$(\sigma = 130,719 \, kg/cm^2) \leq (\sigma' = 3.666,667 \, kg/cm^2) \ldots \textbf{OK}$$

4.2.1   Sidewalk Design

From the results of the calculation of loading on the sidewalk obtained an ultimate vertical load of 12,30 kN and an ultimate moment of 3,075 kN.m. From the results of the calculation of reinforcement obtained reinforcement results:

Reinforcement Position	Main Reinforcement	Shrink Reinforcement
Sidewalk Reinforcement	D16 - 250	D13 - 300
Parafet Reinforcement	D16 - 250	D13 - 300

4.2.2   Vehicle Floor Design

From the calculation results of loading on the vehicle floor obtained from the combination of **Kuat 1**, it obtained an ultimate support moment of 117,947 kN.m and an ultimate mid-span moment of 103,077 kN.m. From the results of the calculation of reinforcement obtained reinforcement results:

Reinforcement Position	Main Reinforcement	Shrink Reinforcement
Support Moment	D19 - 175	D16 - 250
Mid-Span Moment	D19 - 150	D16 - 200
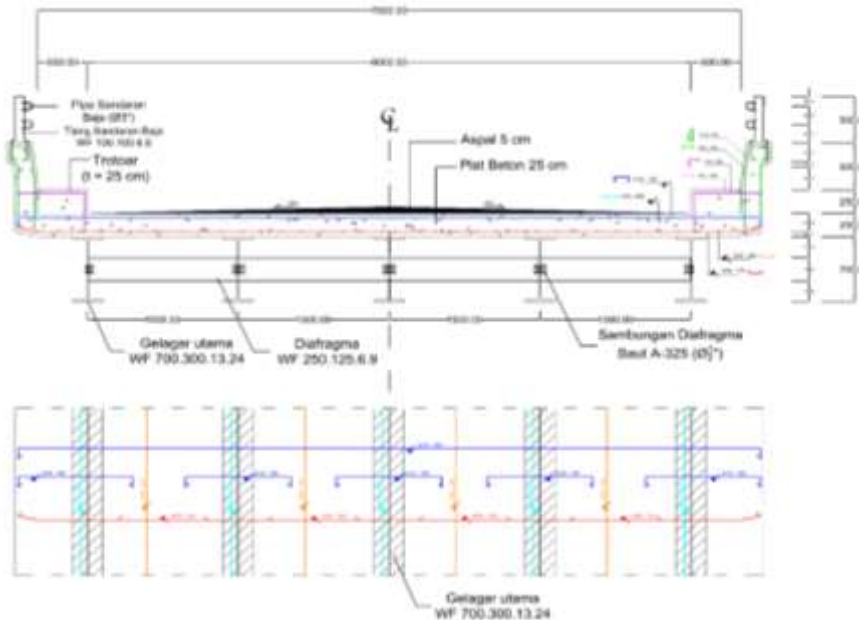
Figure 4.1 Details of Reinforcement of Vehicle Floor Slabs, Pavement and Parapets

4.2.3 Bridge Girder Design

In girder design the profile assumptions are 1350.800.100.130 BJ 55. From the calculation results of loading on the bridge girder obtained from the combination of **Kuat 1**, the ultimate moment is 15.447,29 kN.m and the ultimate shear is 1.927,77 kN.

1. Check assumed profile

Check back on profile:

$$\left(Z_{x\,profil} = 114.675.750,00\ mm^3\right) > \left(Z_{x\,perlu} = 37.676.318,41\ mm^3\right)\ldots \textbf{OK}$$

2. Check the Stress Occurred at the Girder

Check the girder stresses before and after the composites:

 a. Stress that occurs in the main girder before the composite

  • On the upper edge of the flange above the profile

$$f_{sa} = \frac{M_{max} \times \frac{hw}{2}}{I_x} = \frac{6.954,58 \times 10^6 \times \frac{1.022,15}{2}}{3.890.594,07 \times 10^4} = 91,36\ \text{MPa} < f_y = 410\ MPa\ \textbf{OK}$$

  • On the lower edge of the flange above the profile

$$f_{sb} = f_{sa} = 91,36\ \text{MPa} < f_y = 410\ MPa\ \ldots \textbf{OK}$$

 b. Stress that occurs in the main girder after the composite

Stress that occurs in the composite girder:

  • At the upper edge of the concrete slab

$$f_{ca} = \frac{M.y_{tc}}{n.I_{tr}} = \frac{(15.447,291 \times 10^6) \times (82,503 \times 10)}{7,769 \times (6.618.868,105 \times 10^4)} = 24,78\ \text{MPa} \quad < \quad f_c = 30\ MPa\ \ldots$$

OK

  • At the bottom edge of the concrete slab

$$f_{cb} = \frac{M.y_{ts}}{n.I_{tr}} = \frac{(15.447,291 \times 10^6) \times (57,503 \times 10)}{7,769 \times (6.618.868,105 \times 10^4)} = 17,27\ \text{MPa} < f_c = 30\ MPa\ \textbf{OK}$$

  • On the upper edge of the upper flange WF profile

$$f_{sa} = \frac{M.y_{ts}}{I_{tr}} = \frac{(15.447,291 \times 10^6) \times (57,503 \times 10)}{(6.618.868,105 \times 10^4)} = 134,20\ \text{MPa} < f_y = 410\ MP\ \textbf{OK}$$

  • On the lower edge of the upper flange WF profile

$$f_{sc} = \frac{M.y_{bs}}{I_{tr}} = \frac{(15.447,291 \times 10^6) \times (77,497 \times 10)}{(6.618.868,105 \times 10^4)} = 180,86\ \text{MPa} < f_y = 410\ MPa \ldots \textbf{OK}$$

3. **Girder's deflection check**

$$(\delta_{maks} = 124{,}04 \text{ mm}) < (\delta_{izin} = 125 \text{ mm}) \qquad \dots \textbf{OK}$$

4. **Girder's Nominal Moment Check**

   a. Nominal bending strength of the girder before the composite

   $$M_u = 6.954{,}58 \text{ kNm} < \phi M_n = 57.778{,}94 \text{ kNm} \dots\dots\textbf{OK}$$

   b. Nominal bending strength of the girder after the composite

$$\phi M_n = 0{,}9 \times 82.049{,}682 = 73.844{,}714 \text{ kNm} > M_u = 15.447{,}291 \text{ kNm} \quad \textbf{OK!}$$

   c. Nominal bending flexural lateral girder

   $L_b = 7.500 \text{ mm} \geq L_p = 773{,}939 \text{ mm}$

   If $L_b \geq L_p$ (3rd Condition, Article F2.2 SNI 1729-2015), then the value of Fcr is calculated

$$f_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2} \sqrt{1 + 0{,}078 \frac{Jc}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2} = 107.885{,}71 \, MPa$$

$$M_n = S_x \times f_{cr} = 57.638{,}43 \times 107.885{,}71 = 6.218.362.943.652 \, Nmm$$

Because the Mn value of the nominal flexural lateral bending torque is smaller than the nominal bending of the yield (plastic moment), then the nominal Mn flexural strength of the yield (plastic moment) is used.

5. **Shear Connector**

   From the calculation results used a shear connector in the form of Stud with type ESC11-22-125 (L = 125 mm, D = 22 mm) with s = 200 mm.

6. **Bolt Connection of Main Girder to Main Girder**

   From the calculation results obtained on the flange mounted 44 2" bolts arranged in 4 rows to 11 pieces per row on the flange connecting plate with dimensions (3.310×800×70) mm. Whereas 36 bolts on the web were installed 2" arranged in 6 rows to 6 bolts per row on the web joint plates with dimensions (900×1.810×80) mm.
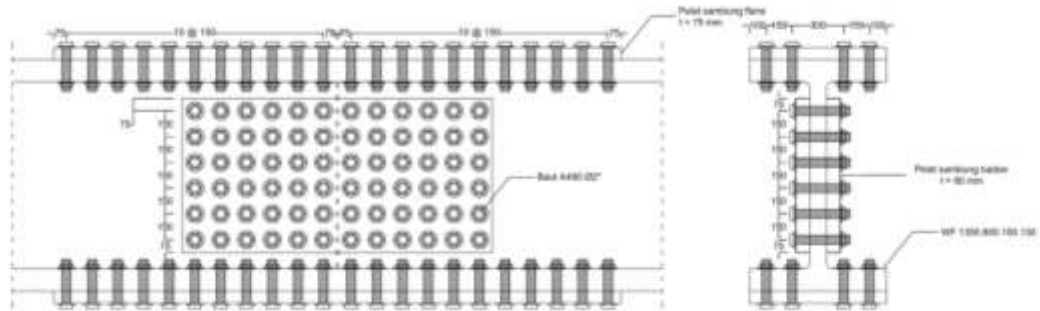
Figure 4.2 Bolt Connection of Main Girder to Main Girder

7.   Bolt Connection of Main Girder to Diaphragm Beam

From the calculation results installed 2 1/2" bolts arranged in 2 rows to 1 piece per row on the body joint plates with dimensions per side **(100 × 50 × 6) mm** or the total dimensions **(100 × 100 × 6) mm**.
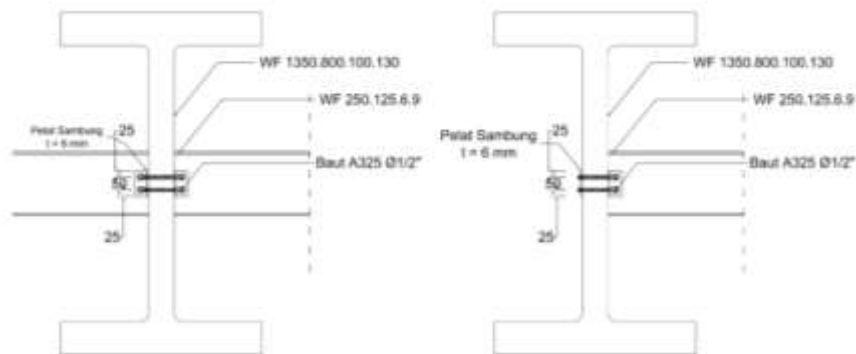

Figure 4.3 Bolt Connection of Main Girder to Diaphragm Beam

4.3   Bridge Substructure Design

The design of substructure of the bridge consists of calculating the forces acting on the pile, calculating the axial and lateral bearing capacity, calculating the efficiency of the pile and pile settlement.

4.3.1   Calculating the Forces Acting on the Pile

From all loading combinations the force distribution is carried out on the pile and the maximum is obtained on the **Kuat 1** combination. The maximum force that occurs on the pile: Maximum vertical force (V) = 114,125 tons; Maximum axial force (P) = 114,694 tons; and Maximum horizontal force (H) = 6,204 tons.

4.3.2   Foundation Design

The type of foundation used is Spun Pile Class C with a diameter of 60 cm with a depth of 30 m piling. Calculation of end bearing capacity of the method by Mayerhof (1976) obtained Qb = 342,12 tons and from the method by Kazuto Nakazawa obtained Qb = 247,12 tons. Of the two methods used the smallest, namely Qb = 247,12 tons. Calculation of the frictional bearing capacity of the clay layer using the α method and obtained Qs = 68,424 tons and the sand layer using the method by Mayerhof (1976) obtained Qf = 74,08 tons. From the calculation of the bearing capacity, the allowable bearing capacity is obtained:

$$Q_{allow} = \frac{Qb}{SF_1} + \frac{Qf + Qs}{SF_2} - W_p = \frac{247,12}{3} + \frac{74,08 + 68,424}{2,5} - 12,27 = 116,37 \ tons$$

Calculation of the efficiency of the Group Pole on sandy soil is taken from the graph by O'Neill (1983). From the graph in Figure 4.4 we get an efficiency value of $E_g = 1.20$, taken $E_g = 1.00$.
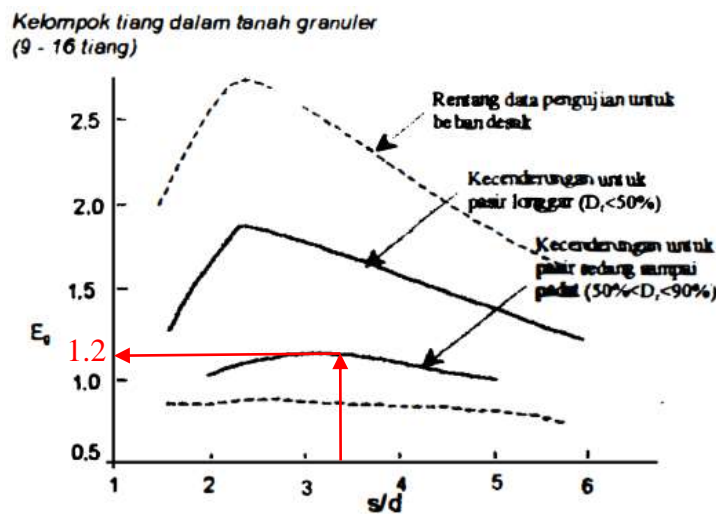

Figure 4.4 Pole Group Efficiency from the Pole Group Test Model in Granular Soil

Then the effective carrying capacity of a single pole is obtained with Eg = 1.00 then $Q_{eff} = 1,00$ x 123,52 tons =B 123,52 tons. Control of maximum force:

Control: $Q_{group} = 116,37$ tons > P = 114,125 tons …**OK!**

The lateral bearing capacity of the pile is calculated using the Brom method obtained Q_L = **23,76** ton. Then divided by the SF number = 3,0 so that $Q_{allow} = \frac{Q_L}{SF} = \frac{23,76}{3,0} = 7,92$ tons. Controls are performed on H_{work} :

$$Q_{allow} = 7,92 \; ton \geq \; H_{work} = 6,20 \; ton\dots \textbf{OK!}$$

Calculation of the settlement of group piles on coarse grained soil (sand or gravel) according to Schultze and Sheriff (1973) can be done by assuming that the group piles are Equivalent Raft Foundation. From the calculation results obtained the settlement S = 11,73 mm, where the value is below S_{Allow} = 50 mm (For Bridge Structures). S_{Allow} obtained from the book Foundation Design: Principles and Practices 2nd Ed. By Donald P. Coduto.

4.3.3   Calculation of Reinforcement of Abutment and Wing Wall

From the results of the calculation of the loading on the abutment and wing wall, the reinforcement results are obtained:

Reinforcement Position	Main Reinforcement	Shrink Reinforcement	Shear Reinforcement
Section I-I Reinforcement	D16 - 100	D13 - 200	D13 - 150
Section II-II Reinforcement	D19 - 75	D13 - 125	D13 - 75
Section III-III Reinforcement	D25 - 150	D19 - 200	D13 - 150
Section IV-IV Reinforcement	D25 - 150	D19 - 200	D13 - 150
Wing Wall Reinforcement	D25 - 150	D19 - 200	-

4.3.4   Oprit Design

From the calculation results obtained $H_{landfill}$ is bigger than H_{cr}:

$$H_{landfill} = 2,0 \; m \leq H_{cr} = 1,68 \; m\dots \textbf{OK}$$

Used reinforcement on subgrade using Galam Ø10 cm with a depth of 5 m S = 35/35 cm. Obtained allowable bearing capacity (qu_{allow} = 4.90 Tons> qu_{total} = 4.85 Tons). Then used a reinforcement in the form of Geotextile with type 150 GR Woven Polypropylene (Tensile Strength = 32-36 kN) at the base of the

landfill and galam pile Ø10 cm with a depth of 5 m S = 35/35 cm. Using GeoSlope/W obtained $SF_{Slope} = 8,144 > SF_{Allow} = 1,5$.

4.3.5 Cost Budget Plan

From the design drawings, it can be calculated the volume of each bridge material which is grouped into several divisions according to the type of work in accordance with Bina Marga standards. From the calculation results, the cost budget plan for the construction of the bridge on the Seluang River-1 is Rp 8.990.566.000,00.-

5   Closing

5.1   Conclusion

Based on the results of the design it can be concluded that:

1. Bridge supestructure design:

   a. The bridge is designed as a bridge type composite structure class B with a span of 30 meters and a bridge width of 7, where 2 x 0.5 m are sidewalks on the right and left side and 6 m is a traffic lane (two-way without a median).

   b. The concrete compressive strength for superstructures in the form of floor plates and parafets uses K-350 concrete quality (*fc'* 30 MPa).

   c. The quality of steel used in the design of the girder uses a profile steel of WF 1350.800.100.130 with BJ 55 steel quality (*fy* 410 MPa).

   d. Floor slabs with a thickness of 250 mm with details of reinforcement D19-200 mm for the main mid-span reinforcement and D19-150 mm for the main support reinforcement, where D16-200 mm for shrink reinforcement on the vehicle floor.

   e. The sidewalk is designed with a thickness of 250 mm with details of reinforcement D16-250 for the main reinforcement and installed D13-300 mm for shrink reinforcement on the sidewalk, and installed reinforcement D16-250 mm for the main reinforcement and installed D13-300 mm for shrink reinforcement on the bridge parafet.

   f. Composite girder with WF profile 1350.800.100.130 fulfills nominal bending requirements:

$\emptyset M_n = 73.844,714$ kNm $\geq M_u = 15.447,291$ kNm.

g. Deflection that occurs in the girder $\delta_{max} = 124,04$ mm $\leq \delta_{izin} = 125$ mm

h. Shear connector using stud type ESC-11-22-125.

2. Bridge subtructure design:

a. The bridge abutment was designed to have a height of 3,50 m, width 3,2 m and a length of 8,5 m.

b. The concrete compressive strength subtructures in the form of abutments uses K-350 concrete quality (*fc'* 30 MPa).

c. The foundation uses a prestressed concrete pile type Spun Pile class C with a diameter of 600 mm.

d. Results of the calculation of the forces on the pile are obtained:
   - Maximum vertical force   = 114,125 tons
   - Maximum horizontal force   = 6,204 tons
   - Maximum axial force   = 114,694 tons

e. In the abutment using prestressed concrete pile type Spun Pile class C with a diameter of 600 mm depth of 30,00 m. From the calculations obtained $Q_{eff} = 116,37 \; tons \geq Q_{work} = 114,69$ tons (safe).

f. By using a graph by O'Neill (1983), the calculation of the efficiency of group poles on sand soil is 100%.

g. Control of safe abutment conditions for lateral forces:

   $H_{allow} = 7,92$ tons $\geq H_{work} = 6,20$ tons (safe).

h. In the abutment calculation, the following reinforcement is obtained:

Reinforcement Position	Main Reinforcement	Shrink Reinforcement	Shear Reinforcement
Section I-I Reinforcement	D16 - 100	D13 - 200	D13 - 150
Section II-II Reinforcement	D19 - 100	D13 - 150	D13 - 100
Section III-III Reinforcement	D25 - 150	D19 - 200	D13 - 150
Section IV-IV Reinforcement	D25 - 150	D19 - 200	D13 - 150
Wing Wall Reinforcement	D25 - 150	D19 - 200	-

i. In oprit given reinforcement in the form of Geotextile with a type of 150 GR Woven Polypropylene (Tensile Strenght = 32-36 kN) at the bottom of the landfill and Ø10 cm galam piles with a depth of 4 m with S = 35/35 cm. Slope's Safety Factor using the GeoStudio 2012 GeoSlope/W software obtained the most critical SF value of 8,144. The value is greater than the required SF value of 1,5.

3. The design plan is attached in the appendix.

4. The planned budget for the construction of a bridge on the Seluang-1 River is Rp 8.909.485.000,00.-

5.2 Suggestion

Soil investigations should be carried out by taking more undisturbed soil samples to obtain more complete and more accurate laboratory test data.

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