

REDESIGN OF THE SUPERSTRUCTURE DIMENSIONS FOR THE SEI BINGAI I STEEL TRUSS BRIDGE, BINJAI CITY

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Abstract

The Sei Bingai I Bridge in Binjai City is a steel truss bridge that was inaugurated in 1997 and requires a re-design study due to the increase in vehicle activity in the surrounding area. This study aims to determine the superstructure load, steel profile dimensions, and the diameter and number of connection bolts. Primary data were obtained through direct measurements, while secondary data came from literature studies. Loading refers to SNI 1725:2016, steel design to SNI 1729:2020, and concrete to SNI 2847:2019. Internal force analysis was carried out using SAP2000. The design maintains the warren truss type with a span of 55 m and a design width of 7 m. The design results in a 200 mm thick floor slab with D16–200 main reinforcement and D12–200 split reinforcement. The longitudinal girders used WF 700.300.15.28, the transverse girders WF 900.300.18.34, the upper members WF 400.400.45.70, and the lower and diagonal members WF 400.400.30.50. The wind ties used L 250.250.20. The main frame connections used 44 upper member bolts and 28 diagonal bolts with a diameter of 24 mm. The inter-girder connections used eight bolts with a diameter of 22 mm, while the wind ties used four bolts with a diameter of 16 mm. The bolts used A325 quality. The study was limited to the upper structure without considering earthquake loads and construction costs.

Keywords: *Steel Truss Bridge; Redesign; Superstructure.*

INTRODUCTION

The Sei Bingai I Bridge connects Binjai Kota District with West Binjai District. Located on Jalan Manggis, Limau Sundai Village, Binjai City, the bridge crosses the Binge River. Since its inauguration on August 15, 1997, the bridge has served as a vital infrastructure supporting public mobility and economic activity. The presence of Tavip Market in the area has resulted in the movement of both light and heavy vehicles transporting merchandise. This situation served as the background for the study of the loading and dimensions of the bridge's superstructure. The existing structure uses a warren truss with a span length of 55 m, a bridge width of 8.5 m, and a pavement width of 6 m. The redesign retained the truss type and span length, but used a design width of 7 m, including a 0.50 m wide sidewalk on each side. The triangular truss arrangement was retained because of its simplicity and transfer of loads through tensile and compressive forces within the truss (Santoso & Sumaidi, 2021). The research was directed at obtaining the magnitude of the superstructure load, determining the dimensions of the steel profiles, and determining the diameter and number of connection bolts. The references used were SNI 1725:2016, SNI 1729:2020, and SNI 2847:2019. The scope of the study included floor slabs, longitudinal and transverse girders, main frames, wind ties, connections, and ancillary buildings. The substructure, seismic loads, unit price analysis, and cost budget plans were not calculated.

LITERATURE REVIEW

Steel frame bridges and previous research

Steel truss bridges use interconnected trusses to support the load. In a warren truss, the diagonals form a triangular structure, while the top, bottom, and diagonal members carry the axial force. The deck slab receives the vehicle load and distributes it through the girders and main frame (Muna & Rivai, 2023). Previous studies provide references for dimensioning, internal force analysis, and connection inspection. Caley and Trimurtiningrum (2023)

designed the superstructure of the Semanding Tuban Bridge with a 250 mm floor slab, longitudinal girders WF 250 × 125 × 6 × 9, and transverse girders WF 1000 × 350 × 18 × 38. Al Hikmi and Noverma (2024) discussed profile planning and connections of a warren truss bridge based on tensile and compression member inspections. Both studies show that profile determination needs to be accompanied by inspection of elements and connections. Putri and Nindyawati (2024) compared the Warren, Pratt, Howe, and K-truss trusses through analysis of the need-to-capacity ratio, structural weight, and deflection. Firdaus and Pratama (2025) reviewed the tensile-compression capacity, moment, shear force, deflection, and stress ratio of the Marsedan Bridge. The scope of this research serves as a reference in examining element responses, while this research specifically applies redesign to the geometry of the Sei Bingai I Bridge. Nurjanah and Febryanti (2023) also conducted a review of existing bridges on the Soekarno-Hatta Bridge, taking into account the main girders, diaphragms, floor slabs, and deflections. Rudi et al. (2024) calculated the member forces and bolt requirements on the Way Mamua Bridge using the knot point method. Yuntikasari et al. (2021) presented alternative girder dimensions, wind ties, connections, and placement for the Gelatik Bridge.

Basic loading and element inspection

The loading consists of self-weight, additional dead load, traffic load, implementation load, braking force, pedestrian load, and wind. The D lane load includes evenly distributed load (BTR) and concentrated line load (BGT). The T truck load is reviewed as an alternative traffic loading, not working simultaneously with the D load. Dynamic effects are taken into account in vehicle loading. The application of SNI 1725:2016 is the basis for adjusting load parameters in planning (Setiyarto, 2018).

Tension member inspections include gross section yielding, net section failure, block shear, and slenderness. Compression members are inspected for slenderness, flexural and torsional buckling. Flexural members are inspected for yielding, local buckling, lateral torsional buckling, shear force, and deflection. Bolted connections are inspected for tension, shear, bearing, tearing, and block shear (Lesmana, 2021). The basic form of the capacity equation used in the study is as follows.

$$P_n = F_y A_g \quad (1)$$

$$P_n = F_{cr} A_g \quad (2)$$

$$M_n = F_y Z_x \quad (3)$$

$$R_n = F_n A_b \quad (4)$$

Equation (1) is used for tensile yielding, equation (2) for compression, equation (3) for nominal moment at yielding condition, and equation (4) for nominal bolt strength. The design strength is obtained by taking into account the appropriate reduction factor, then compared with the internal force due to loading. Sections that do not meet the requirements are reanalyzed by changing their dimensions.

METHOD

The study used a numerical approach through a superstructure re-design study. Primary data were collected by directly measuring the bridge dimensions using a steel tape measure and documenting field conditions. Secondary data were obtained from books, literature studies, and regulations used as the basis for loading and planning. The research location was the Sei Bingai I Bridge on Jalan Manggis, Limau Sundai Village, West Binjai District, Binjai City, North Sumatra Province. The research phase begins with literature review and data collection, followed by load calculations, profile dimensioning, bolt design, and deflection control. After completing all inspection phases, the design results are presented in working drawings and conclusions. The research flow is presented in Figure 1.

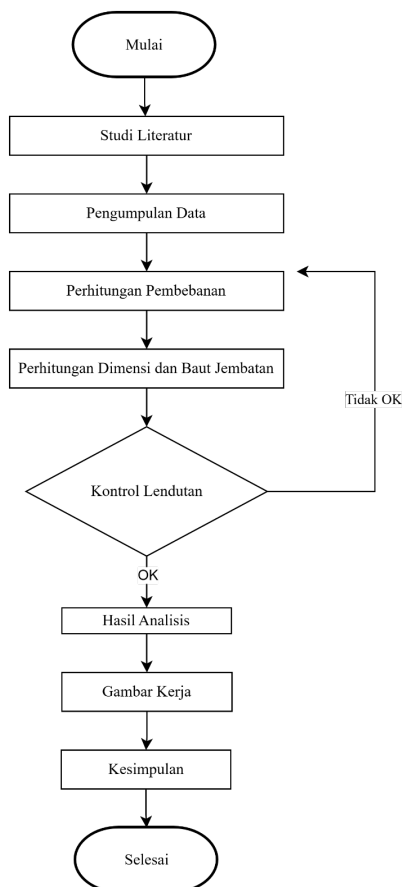


Figure 1. Bridge planning flow diagram.

Data processing begins with the determination of the geometry and dimensions of the initial profile. The calculated loads refer to SNI 1725:2016, then applied to the SAP2000 model to obtain axial forces, moments, and shear forces on the structural elements. Concrete design refers to SNI 2847:2019, while the inspection of steel profiles and connections refers to SNI 1729:2020. The longitudinal girder is modeled as a simple 5 m long beam and the transverse girder is 7 m long.

The analysis results were used to evaluate the suitability of the profile dimensions. If the profile did not meet the requirements, the dimensions were changed and the analysis was repeated. After the profile met the controls, the connections were planned and inspected. Microsoft Excel was used for calculations, SAP2000 for internal force analysis, and AutoCAD 2021 for drawing. The research results represent the design of the superstructure within the specified loading limits, not the results of field bridge load testing.

RESULTS AND DISCUSSION

Technical data and planning of bridge floors

The design geometry uses a span of 55 m, a frame height of 5 m, a traffic lane width of 6 m, and a total width of 7 m. The longitudinal girders have a span of 5 m with a distance between girders of 1.30 m. The transverse girders have a span of 7 m with a distance between girders of 5 m. The floor slab is planned to be 0.20 m thick, the asphalt layer is 0.05 m, and the rainwater puddle is 0.05 m. The sidewalks on both sides have a width of 0.50 m each with a thickness of 0.40 m.

The concrete material uses a compressive strength of 40 MPa and an elastic modulus of 29,725.41 MPa. The planned steel material data uses BJ 41 with a yield stress of 250 MPa, a breaking stress of 410 MPa, and an elastic modulus of 200,000 MPa. The main reinforcement is 16 mm in diameter and the reinforcement is 12 mm in diameter, both with a yield stress of 420 MPa. The unit weight of the reinforced concrete used is 22.88 kN/m³, while the unit weight of asphalt and water is 22.00 kN/m³ and 9.80 kN/m³, respectively.

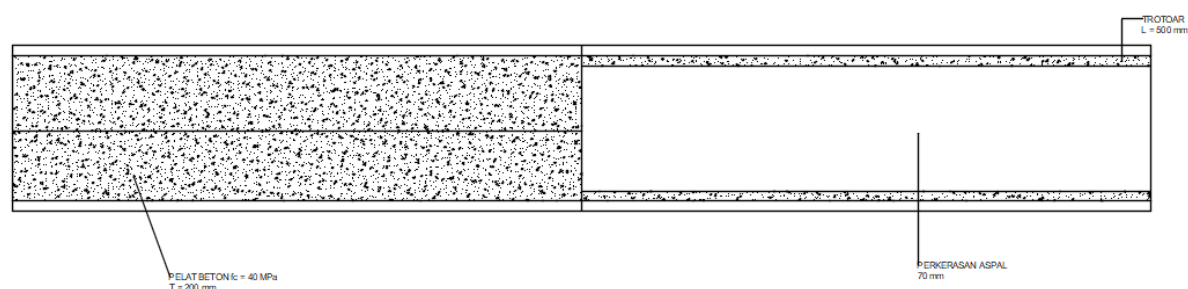


Figure 2. Bridge floor planning.

In the vehicle floor calculation, the plate load per meter of stringer length was recorded at 7.733 kN/m. The additional dead load consisted of asphalt 2.86 kN/m, steeldeck 0.132 kN/m, and rainwater 0.637 kN/m, so that the total in the calculation description was 3.629 kN/m. The planned truck wheel load used was 112.5 kN with a dynamic load factor of 30%. A review of the braking force in the calculation description compared 56.25 kN and 82.337 kN.

The slab reinforcement uses a 40 mm concrete cover. The transverse reinforcement D16–200 produces an installed reinforcement area of 1,005,310 mm², greater than the minimum area of 572,222 mm². The longitudinal reinforcement D12–200 produces an installed area of 565,487 mm², greater than the required 524.4 mm². Thus, the design results use D16–200 main reinforcement and D12–200 split reinforcement in a 200 mm thick slab. Punch shear control uses an effective height of 152 mm, a critical perimeter of 2,608 mm, and a critical area of 396,416 mm². The calculation description provides a factored shear force of 202.5 kN, a nominal concrete shear strength of 827.361 kN, and a design shear strength of 620.52 kN. Based on the comparison reported in the study, the slab meets punch shear control.

Sidewalks and railings

The area of one sidewalk area of 27.5 m² produces a pedestrian load of 4,422 kN/m². The abutment posts are planned to be 1,070 mm high with a distance of 2 m. The abutment pipe uses a 4 inch diameter with a thickness of 2.9 mm, while the posts use a WF 125.60.6.8 profile. The load on the abutment is calculated as a horizontal and vertical force of 0.75 kN/m. These complementary components are calculated together with the bridge floor planning.

Longitudinal girder

The longitudinal girders used the WF 700.300.15.28 profile and were reviewed in pre- and post-composite conditions. The modeled span was 5 m long. Calculations recorded a dead load of 10.939 kN/m before the composite and 21.529 kN/m after the composite. The live loads reviewed included BTR 23.4 kN/m, BGT 67.987 kN, factored truck load 292.5 kN, and implementation load 2.5 kN/m. The combinations and analysis results are presented in Tables 1–4.

Table 1. Combination of longitudinal girders before composite.

Type	Burden
COMB G. Elongated Steel	Ms

Table 2. Longitudinal girder loading combinations.

Type	Burden
COMB 1	DL + LL (BTR+BGT+PLK)
COMB 2	DL + LL (T + PLK)

The SAP2000 output results for two loading combinations are shown in Figure 3. The diagram displays the distribution of shear forces, moments, and deflections in the elements under consideration.

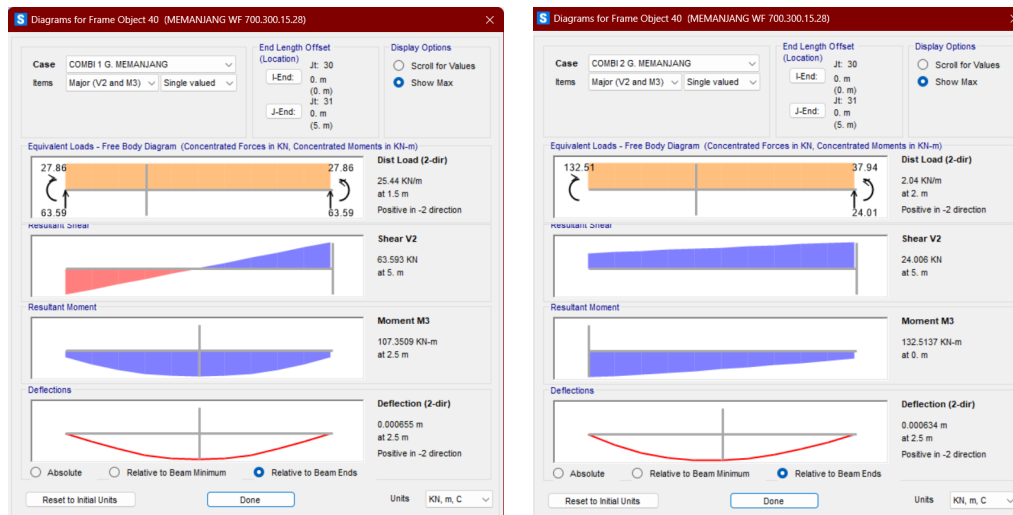


Figure 3. Output longitudinal girders: COMB 1 (left) and COMB 2 (right).

Table 3. Results of longitudinal girder analysis before composite.

Output case	V2 (kN)	M3 (kN)
COMB G. STEEL	7.11	45,194

Table 4. Results of longitudinal girder analysis after composite.

Output case	V2 (kN)	M3 (kN-m)
COMB 1	63,593	107,350
COMB 2	24,006	132,513

The first combination produces a shear force of 63.593 kN, while the second combination produces a moment of 132.513 kN-m. The profile is stated to be compact at the flange and web. The nominal moment calculation yields 1,836.055 kN-m and a nominal shear strength of 1,467 kN. The study's inspections indicate that the control of flexure, lateral torsional buckling, shear, and deflection meets the requirements. The specified deflection limit for this girder is 0.013 m.

Transverse girder

The transverse girder uses WF 900.300.18.34 on a span of 7 m. The profile weight is 286 kg/m, the cross-sectional area is 364 cm², and the moment of inertia of the main axis is 498,000 cm⁴. The dead load before the composite is 41.976 kN/m and after the composite is 82.716 kN/m. The loading also includes BTR 69.55 kN/m, BGT 68.6 kN/m, truck load 292.5 kN, and implementation load 2.5 kN/m.

Table 5. Combination of transverse girders before composite.

Type	Burden
COMB G. TRANSVERSAL STEEL	Ms

Table 6. Combination of transverse girders after composite.

Type	Burden
COMB 1	DL+PLL
COMB 2	DL + LL (BTR+BGT)
COMB 3	DL + LL (T1)
COMB 4	DL + LL (T2)

The internal force output in Figure 4 shows the response of the transverse girder. The values before and after the composite remain presented in the original analysis results table.

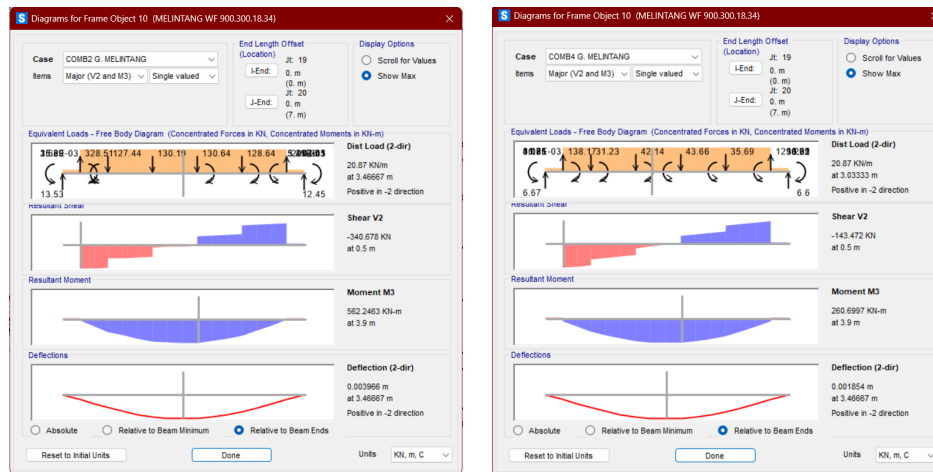


Figure 4. Transverse girder output: COMB 2 (left) and COMB 4 (right).

Table 7. Results of transverse girder analysis before composite.

Output case	V2 (kN)	M3 (kN)
COMB G. STEEL	38,605	91,026

Table 8. Results of transverse girder analysis after composite.

Output case	V2 (kN)	M3 (kN-m)
COMB 1	84,588	154,990
COMB 2	340,678	562,246
COMB 3	340,678	562,246
COMB 4	143,472	260,699

COMB 2 was designated as the determining combination in the study, with a shear force of 340.678 kN and a moment of 562.246 kN-m; COMB 3 in the table has the same values. The reported design flexural capacity is 2,749.68 kN-m and the nominal shear strength is 2,278.80 kN. The profile is stated to meet the flexural, shear, and deflection control with a deflection limit of 0.019 m.

Main frame: upper stem and lower stem

The main frame was designed by maintaining the warren truss configuration. The upper truss elements used WF 400,400,45,70, while the lower truss and diagonals used WF 400,400,30,50. The profile selection followed the results of the axial force in the model and the capacity check of each element. The truss arrangement and truss numbering are shown in Figure 5.

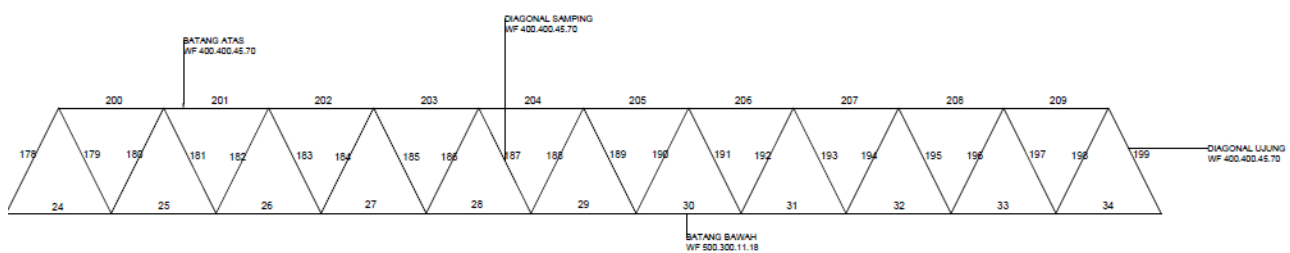


Figure 5. Main truss structure.

Top horizontal bar

Bar 205 is a compression member which is considered with an axial force P_u of 12,034.209 kN. The WF 400.400.45.70 plan profile is checked for cross-sectional slenderness, flexural buckling and torsional buckling. On slenderness examination, the wings and body were declared compact. The effective length factor used in the calculation is $K = 1$ according to the basic assumptions in the research.

For torsional buckling control, the calculation recorded a critical stress of 209 MPa, a nominal compressive strength of 16,095.09 kN, and a design compressive strength of 14,485.581 kN. These results were compared with the compressive forces obtained from the analysis. Based on the reported control, the upper bar profile met the requirements under the conditions reviewed. The output axial force of the upper bar is shown in Figure 6 on the left.

Lower horizontal bar

Bar 29 is a tensile bar with an axial force P_u of 2,756.461 kN. The planned profile is WF 400,400,30,50. Inspections were conducted on slenderness and tensile strength, including gross section yield and calculated section failure at the connection area. The design description states that the profile meets the inspections performed. The bottom bar output is shown in Figure 6 on the right.

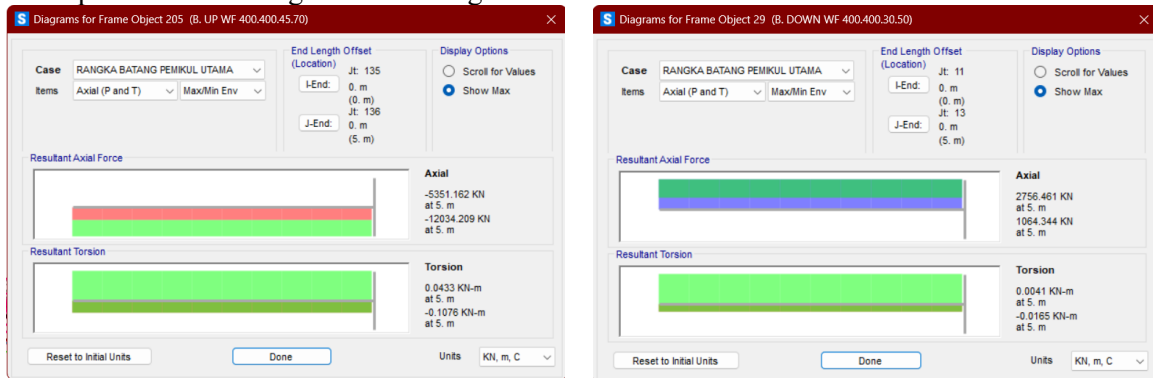


Figure 6. Axial force output of the upper (left) and lower (right) rods.

The difference in forces between the upper and lower members forms the basis for determining different profiles. The upper member is viewed as a compression element, while the lower member is viewed as a tension element. Therefore, the examination is based not only on the magnitude of the axial force but also on the boundary conditions used for each element. The results of selecting these two profiles are retained in the final dimension summary of the study.

Diagonal bars and wind ties

The diagonal members use the WF 400,400,30,50 profile. Member 199 as the end diagonal is reviewed with P_u of 4,560.83 kN. Member 198 works as a tensile diagonal with P_u of 4,430.131 kN, while member 180 is reviewed as a compressive diagonal with P_u of 3,580.278 kN. The output of tensile and compressive diagonal axial forces is presented in Figure 7.

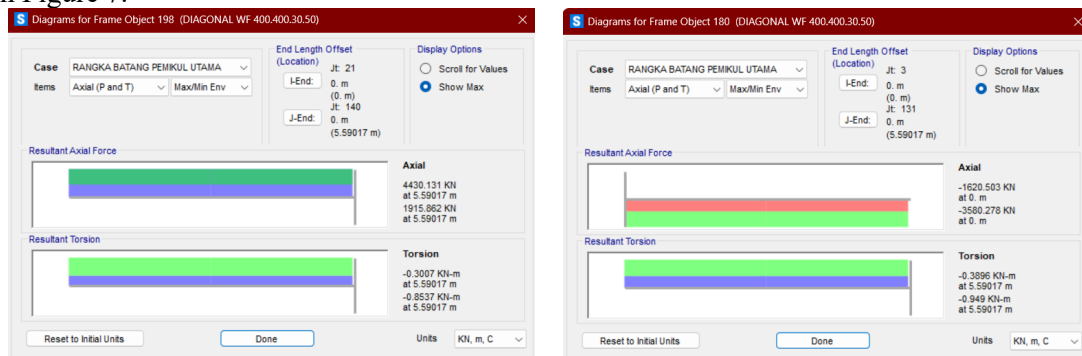


Figure 7. Pull diagonal (left) and push diagonal (right) output.

Slenderness and flexural buckling were checked on the end diagonals. Other diagonals were checked according to the tensile or compressive conditions reviewed in the calculation. Torsional buckling control on the compression diagonal recorded a critical stress of 151 MPa, a nominal compressive strength of 7,981.86 kN, and a design compressive strength of 7,183.674 kN. The study maintained the WF profile of 400,400,30,50 for the diagonal members after the inspection was declared to be in accordance with the requirements.

Wind loading and stiffener profiles

The wind bond design uses a basic speed of $V_{10} = V_B = 90$ km/h, elevation $Z = 10$ m, friction speed $V_0 = 17.6$ km/h, and friction length $Z_0 = 1,000$ mm. The calculated result of the planned wind speed is 101.314 km/h or 28.143 m/s. The wind pressure on the structure consists of a compressive wind of 3.041 kN/m² and a suction wind of 1.521 kN/m², with a total of 4.562 kN/m².

The structural wind force on the area under review was recorded at 114,050 kN. The effect of wind on the vehicle was calculated as a line load of 1.46 N/mm acting at a height of 1,800 mm above the road surface. The wind ties used an equilateral angle profile L 250.250.20. Internal forces were taken from a combination of Strength III and Strength V because both combinations take wind into account.

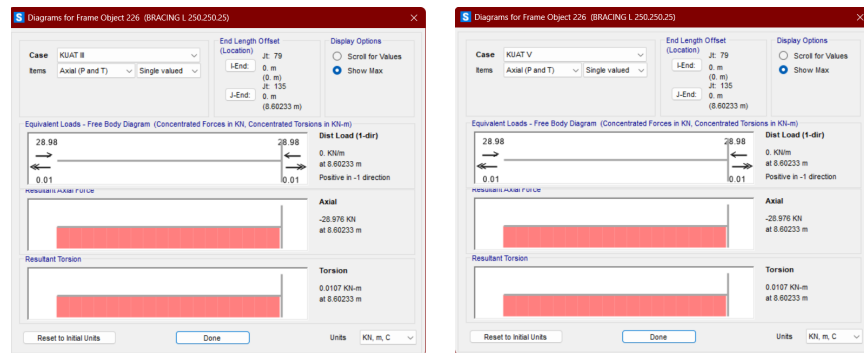


Figure 8. Wind bond output at Strength III (left) and Strength V (right).

The P_u value used to control the wind bond profile is 28.976 kN. The inspection includes slenderness as well as tensile and compressive strength. Based on the planning results, the L 250.250.20 profile is used as the upper wind tie. In the connection planning section, the axial force used was recorded separately at 35.226 kN.

Connection of the gusset plate and the main frame

The main frame connection is designed using high-strength A325 bolts, with threads included in the shear plane. The bolt data used includes a nominal tensile strength of 620 MPa and a nominal shear strength of 372 MPa. The joint plate uses a yield stress of 250 MPa, a breaking stress of 410 MPa, and a modulus of elasticity of 200,000 MPa. One of the knot points on the upper frame is selected for the connection design. The specific calculation of the gusset plate uses a plate width of 1,324 mm, a plate thickness of 40 mm, and two shear planes. The main frame bolts are set at 24 mm in diameter. Figure 9 shows the arrangement of the gusset plate and diagonal joints retained from the research design drawings.

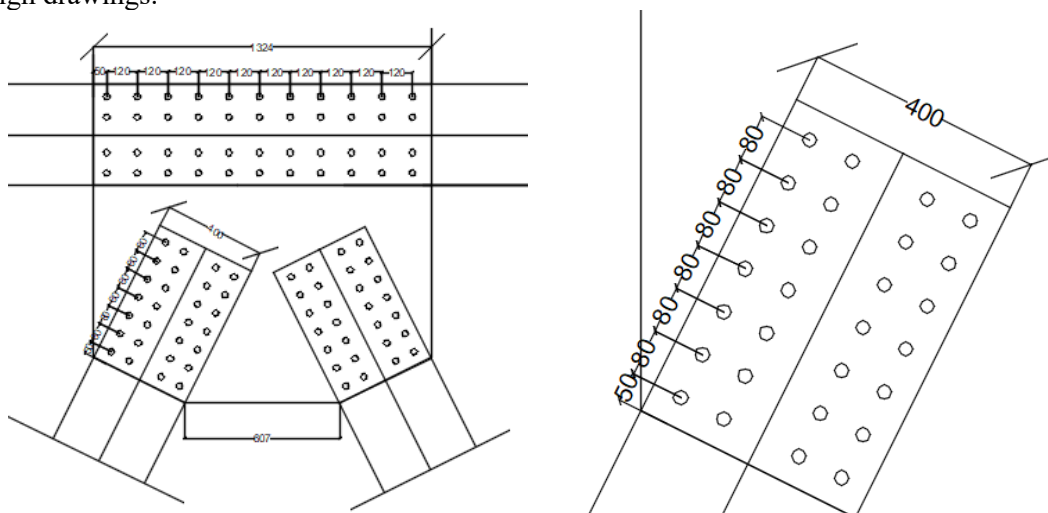


Figure 9. Main frame gusset plate (left) and diagonal joint (right).

Top and diagonal bar bolts

The upper bar connection uses a WF 400.400.45.70 profile and a design force P_u of 5,351.16 kN in the connection calculation section. The nominal shear strength of one bolt is recorded at 168.144 kN. The calculation of the number of bolts produces 42.43 pieces, so 44 bolts with a diameter of 24 mm are used. The edge distance of the plate used is 50 mm and the distance between bolts is 120 mm. Subsequent inspections include supports, bolt hole tears, and connected plate elements.

On the diagonal, WF 400,400,30,50 was used. The connection calculation section reviewed the P_u force of 4,424.416 kN and 3,585.041 kN on the discussed members. The final results of the study determined 28 A325 bolts with a diameter of 24 mm for the diagonal connection. The edge distance used was 50 mm and the distance between bolts was 80 mm. The connection control included shear, tensile, bearing, tearing, as well as block cross-sectional and shear strength.

Block shear inspection is carried out by reviewing the gross area of the shear plane, the net area of the shear plane, and the net area of the tension plane. In the gusset plate, the failure plane used in the calculation is shown in Figure 10. This control is part of the inspection of the connected plate elements, in addition to the inspection of the bolts themselves.

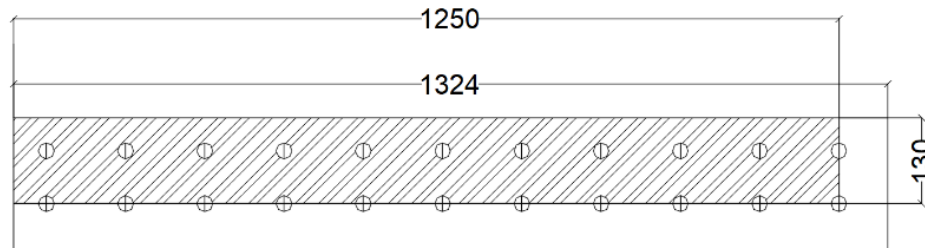


Figure 10. Shear plane of gusset plate block.

Inter-girder connections and wind ties

The longitudinal girder connection with the transverse girder uses A325 bolts with a diameter of 22 mm, a bolt area of 380 mm², and a hole diameter of 24 mm. The connecting plate is an L 100.100.10 angle profile with a thickness of 10 mm. The planning results specify eight bolts. The bolt distance from the edge of the plate is taken as 30 mm, while the distance between bolts is 70 mm. The arrangement of the angles and details of the wind tie connection are presented in Figure 11.

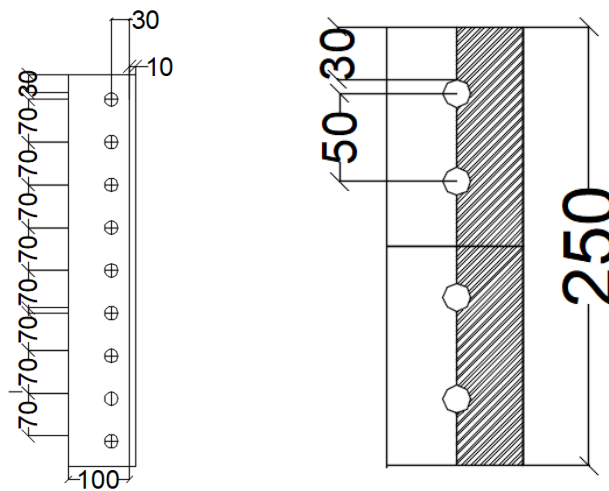


Figure 11. Inter-girder connection angle (left) and wind tie detail (right).

The inter-girder connection inspection included bolt shear strength, bearing strength, plate tear strength, and block shear strength. The bearing calculation recorded a nominal strength of 216.48 kN and a design strength of 162.36 kN. The calculated number of bolts, 6.8, was rounded up to eight. The inspection of two angle plates resulted in a block shear capacity of 1,356.6 kN, which in the calculation description was compared with 1,100.25 kN and was declared to meet the requirements.

Wind tie bolts and connecting plates

The wind tie connection is planned to use A325 bolts with a diameter of 16 mm and a 20 mm thick joint plate. The bolt area used is 201 mm². Calculations recorded a nominal shear strength of one bolt of 74.795 kN and a nominal bearing strength of 314.88 kN. The design strength of the bolt used is 56.096 kN. The axial force in the planned connection section is recorded at 35.226 kN. The calculation results require 0.628 bolts, but the number used in the design is four bolts. The edge distance is set at 30 mm and the distance between bolts is 50 mm. The next inspection includes the plate tear strength and block shear strength. In the block shear control, the calculation description recorded a nominal strength of 246 kN for the fracture state and 150 kN for the yield state. From the smallest value obtained, the design strength is 112.5 kN for one plate or 225 kN for two plates.

Shear connectors on composite girders

The girder design also includes shear connectors. The type used is a stud connector with a diameter of 19 mm and a length of 120 mm, with BJ 41 steel grade and a breaking stress of 410 MPa. The connector requirement is reviewed through the longitudinal shear force and nominal strength of the stud with reference to the steel anchors section in SNI 1729:2020. The discussion of the girder thus covers the pre-composite state, the post-composite state, and the connection between the steel and concrete.

Final result of superstructure planning

The design results relate vehicle floor loading, girder response, frame axial forces, and connection requirements. The longitudinal and transverse girders are examined in their pre- and post-composite states, while the main frame is reviewed for tensile or compressive forces in selected members. Wind tie analysis completes the lateral force review. The three-dimensional model used in the study is shown in Figure 12.

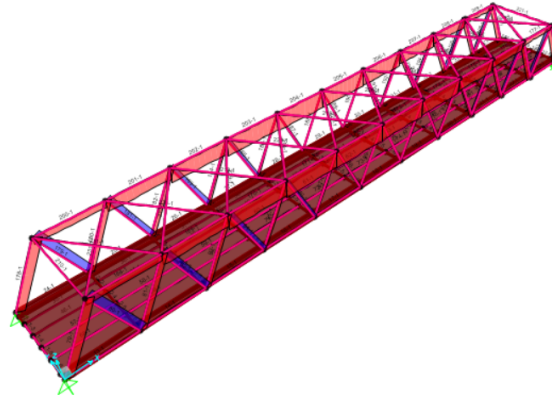


Figure 12. Bridge structure model in SAP2000.

The dimension selection was followed by the design of the joint plate connections, inter-girder connections, and wind ties. These results answered the research objectives regarding the magnitude of the load, steel profile, and the diameter and number of bolts. As a limitation of the research, the results obtained apply to the scope of the superstructure and the loads reviewed. The research did not include analysis of the substructure, earthquake loads, or proof of construction cost efficiency.

CONCLUSION

The redesign of the Sei Bingai I Bridge retained the warren truss structure with a span of 55 m and a design width of 7 m. The superstructure loads included self-weight, additional dead load, traffic, implementation, braking force, pedestrians, and wind. The longitudinal girder analysis results in a shear force of 63.593 kN at COMB 1 and a moment of 132.513 kN-m at COMB 2. For the transverse girder, the combination of determinants used resulted in a shear force of 340.678 kN and a moment of 562.246 kN-m.

The floor slab is planned to be 200 mm thick with D16–200 main reinforcement and D12–200 split reinforcement. The final profiles used are WF 700,300,15.28 for the longitudinal girders, WF 900,300,18.34 for the transverse girders, WF 400,400,45.70 for the upper members, WF 400,400,30.50 for the lower and diagonal members, and L 250,250,20 for the upper wind ties. These dimensions are the design results that are maintained in the research conclusions. The main frame connection uses 24 mm diameter A325 bolts, 44 on the upper beam and 28 on the diagonal. The longitudinal girder connection with the transverse girder uses eight 22 mm diameter A325 bolts. The wind tie connection uses four 16 mm diameter A325 bolts. Subsequent planning needs to be done with precision in selecting dimensions and loading to avoid excessive or insufficient sizes. The distribution of the superstructure load can be the basis for developing studies on the bridge's substructure.

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