The effectiveness of the using a K-truss type on the Patikraja bridge

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Abstract- The Patikraja Bridge in Banyumas Indonesia is a truss bridge of the Pratt type, measuring 127 meters in length and 5 meters in width. It was built for trains during the Dutch colonial period and is currently being transformed into a public route for automobiles. As an alternate design, the bridge is redesigned with K-type steel trusses. The purpose of this paper is to explore the design of truss bridge including the dimensions of the steel profile that will be used. LRFD (Load Resistance Factor Design) is used in conjunction with SNI 1726:2016 loading (Indonesian National Standard). The profile of the wind bracing was calculated using the 500x200x10x16 and 500x500x25x40 profiles. The 500x500x25x40 H profile is used for the top side bars, diagonal side bars, and side bars. Steel profiles H 450x200x9x14 for the longitudinal girders and H 700x300x13x24 for the transverse girders were used. Bolt connection with bolts measuring 112 inches. The bridge deflection is 36 mm. The dimensions of the plate were determined to be 7x45x0.25m. The bridge's backrest pipe is made of 3 inch pipe. 725x740 mm elastomeric bearing. The slab measures 7 meters wide by 6 meters long. The body of the abutment is 2 meters thick and 8 meters long. The abutments for the pilecaps are 8 m broad and 9 m long. The wing wall measures 5.5 meters in width and 6.75 meters in height. For example, if the foundation on the abutment is 0.5 m in diameter and 10 m in length, there could be up to 18 foundations.

Index Terms- truss bridge, redesign, steel profile, K-truss type

I. Introduction

The continuing movement of people and their vehicles, along with increased criteria for the design and construction of their truss bridge structures, greatly exceeds the accumulated construction obtained in the numerous research.

Research by [1] is to explore the subject of optimizing the carbon fiber content distribution in truss bridge structural elements of a plane, statically determinate regular elastic truss construction. A version of the genetic algorithm is proposed, along with a method of mathematical induction. As an optimality criterion, a cost minimization criterion for structural materials is applied [1].

Ultimate load tests on a steel truss bridge transferred to the structural laboratory were conducted by [2] as part of an effort to better understand the behavior of historic steel truss bridges. The first ultimate load test was conducted with the bridge in its current configuration, without any retrofits. The work by [3] divides Louisville truss bridge constructions with numerous joints into several substructures, including interior and external substructures, and then applies a damage identification approach to each substructure. To illustrate various aspects of this approach, numerical validations are performed by [3] on an interior substructure with more joints than the outward substructure. The dead load also can be assumed to be applied in the defined location as depicted in Fig.1. The works by [4] covers an unique composite tubular truss bridge with a concrete slab and rectangular chords filled with concrete. With a concrete slab plus truss system and reinforced connections with concrete and Perfobond Leiste rib, the double composite truss bridge proved to be a reasonably adequate option in the negative moment area [4].

The contribution research provided by [5] for the computerized design and automated production of a greened timber truss bridge for pedestrians in the vicinity of the University of Natural Resources and Life Sciences in Vienna using traditional construction techniques. Their study defined the design concept's boundary conditions and examined the spatial situation and pedestrian movements in the study region of truss bridge.

The research concerning truss bridge structural design are limited, contrary to the importance of their structures due to their rapid construction, comparing to the other bridge structures such suspension bridge, cable bridges and also prestressed and girder bridges. This paper is amed to explore the design of steel truss bridge structures especially Pratt type.

II. LITERATURE REVIEW

A. Definition of bridges

A bridge, in general, is a structure that connects two sections of a road that are separated by obstacles such as deep valleys, river channels, lakes, irrigation canals, rivers, railroads, and unplanned highways [6].

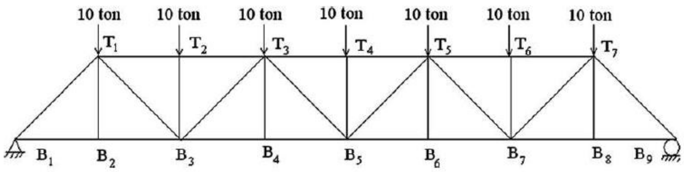


Fig. 1 Assumed dead load defined in joint location in Louisville Bridge
Source: [3]

B. Finite element application to truss bridge design

Reference [7] addresses the truss bridge design using the statics module utilized to conduct FEA analysis of the structural performance in order to optimize the structure's load-carrying capability.

Along with using the FEA simulation as a design tool, students may choose to conduct theoretical calculations for a complete truss analysis and find the bar that is subjected to the largest internal force. This, too, is based on FEA and requires some coding. Its primary steps are as follows: (a) Discretization, for which the simplest line element, namely one element for each bar, is chosen. (b) Determination of the displacement function, which will be linear for simplicity; (c) Derivation of the element stiffness matrix *[k]* and equation for each bar. Eq. (1) is the element stiffness equation for a generic bar oriented arbitrarily as shown in Fig. 2, where θ is positive when measured counterclockwise from x to x' and c and s are abbreviations for the cosine and sine of θ. Additionally, u and v denote the two components of global displacement [7].

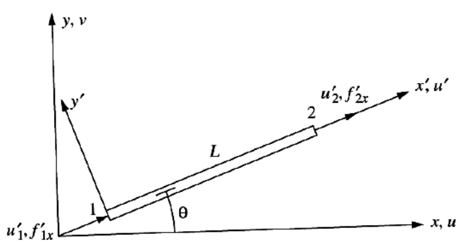


Fig. 2 A truss element oriented randomly in x-y plane
Source: [7]

$$\begin{Bmatrix} f_{1x} \\ f_{1y} \\ f_{2x} \\ f_{2y} \end{Bmatrix} = [k]\{d\} = \frac{AE}{L}\begin{bmatrix} c^2 & cs & -c^2 & -cs \\ & s^2 & -cs & -s^2 \\ sym & & c^2 & cs \\ & & & s^2 \end{bmatrix}\begin{Bmatrix} u_1 \\ v_1 \\ u_2 \\ v_2 \end{Bmatrix} \quad (1)$$

The *f's* in the preceding equation represent the force components applied at either end in the global coordinate system. Additionally, A denotes the cross-sectional area, E denotes the Young's modulus, and L denotes the length. The following steps are as follows: (d) Assembling all element stiffness equations using the direct stiffness approach, as well as enforcing load and boundary conditions [7].

C. The design and analysis development of truss bridge

The development in material of truss bridge have been researched in last decade. For example, reference [1] have presented computational approaches for optimizing the distribution of CFRP within composite elements of truss bridge constructions. To address the issue, a variation of the genetic algorithm was devised in conjunction with mathematical induction approaches. As an optimality criterion, the cost of structural materials is minimized while still meeting stiffness requirements. Based on their modeling results, an optimum design for a composite material pedestrian truss bridge with a span length of 36 m and a pathway width of 3 m has been designed. The use of CFRP in the amount of 7.4 percent of the structure's overall weight resulted in a 9.1 percent reduction in structural cost while still meeting stiffness criteria [1].

Also in evaluation field test such as research by [2] that summarizes the results of ultimate load tests conducted on a steel truss bridge that was brought to the University of Nebraska-structural Lincoln's laboratory. They did two ultimate load tests. From the first test, it was determined that a rupture of a forged tension part results in an extremely brittle mode of failure that occurs without warning. The failed member was refitted, along with other forged members of the truss, and an additional ultimate load test was conducted. They discusses possible methods for retrofitting tension members with forged sections. By retrofitting the truss, the failure was compelled to occur in the truss's top chords, which are compression elements. The truss's compression elements serve as beam–columns. As a result, they are prone to out-of-plane bending from the moment the load is applied. As a result, the failure mechanism is more ductile than that of forged tension members, and enough warning occurs prior to the bridge failing [2].

Also for damage research was presented by [3]. The damage percentage of joints in truss bridges is estimated using a neural network-based system identification approach. The numerical examples were a simple truss and a genuine truss bridge. Substructural techniques were applied to decrease unknown parameters in a genuine truss structure.

They also conclude that the proposed method pinpoints the position and severity of deterioration in truss bridge joints. And the substructuring technique reduced the number of unknown damage characteristics to estimate. An MLP network architecture can locate and assess problems in truss bridges. It was discovered that the average error for testing data set employing five modes was under 1%, demonstrating the method's suitability for large structural systems identification. They also stated that five mode parameters are necessary for truss bridge damage detection and said that their approach is extremely appealing for on-line or real-time structural damage diagnosis [3].

A new type of composite tubular truss bridge was proposed by [4] which have the double composite superstructure. With rectangular tubular chords and a prestressed concrete slab, it should increase local strength and fatigue performance. The post-connected slab system and prestressed tendon are integrated as erection methods to prevent the hogging zone fracture.

Steel–concrete composite trusses bridge have tested by [8] both experiments and FE modeling to study their connection performance. The connection specimens performed satisfactorily under static loading in the experiments. The trial cracking load was 2.21 times the design load, while the ultimate load was 3.87 times the design load. The FE models of the connection specimens agreed well with the experimental results for failure mechanisms and load displacement responses. The validated FE results revealed the stress concentration areas [8].

Experiments were conducted by [9] to determine the mechanical behavior of a steel–concrete connection in a composite truss bridge. They conducted pull-out tests on eight specimens to determine the bearing capability of Perfobond ribs shear connections at various embedded depths. A model test on three composite joint specimens was used to evaluate the connection performance in the composite truss bridge. They concluded that the pull-out and composite joint tests demonstrated that the pull-out specimens and steel–concrete composite junctions exhibited good resistance to an external load. As a result of their findings, Perfobond ribs shear connectors offer superior resistance to and transfer of shear force [9].

While another researcher performed the experiments on a double skin tubular truss bridge subjected to quasi-static monotonic loading (DSTT). The DSTT bridge module was loaded to its serviceability limit, whereas the single DSTT was loaded to failure. They noted that the ductile nature of DSTT results in a negligible stiffness reduction. Typically, failures are limited to specific regions [10].

The study by [11] purposes was to determine the effect of temperature gradients on HRA (horizontal rotation angle) in a steel truss bridge with a two-layer deck. Their results established the presence of considerable temperature gradients in the steel truss bridge, established that thermal load is the primary load affecting HRA, and established which sort of temperature gradient has a significant effect on HRA. They concluded that steel truss bridge has significant vertical temperature gradients between the top and bottom truss members in the side trusses and significant transverse temperature gradients between the bottom truss members in the two side trusses; the transverse temperature gradient between the bottom truss members in the two side trusses has a significant effect on HRA, indicating that reducing this type of transverse temperature gradient is an effective way to reduce HRA response [11].

The work by [12] used finite element analysis to investigate the vibration behavior of a Warren truss bridge. The bridge's model was created in accordance with Indian standards. The bridge in their analysis was constructed as a composite structure to improve its structural qualities. Model investigation of the vibration behavior of this composite structure was performed using ANSYS. The vibration and deformation simulation results were satisfactory. A frequency-based analysis was used to determine the resonance frequency of the bridge under consideration. They found that the first six types of failure have a frequency range of 7.6142 to 15.098 Hz. As a result, 7.6142 Hz was chosen as the bridge's fundamental operating frequency [12].

III. CASE STUDY OF A K-TRUSS BRIDGE

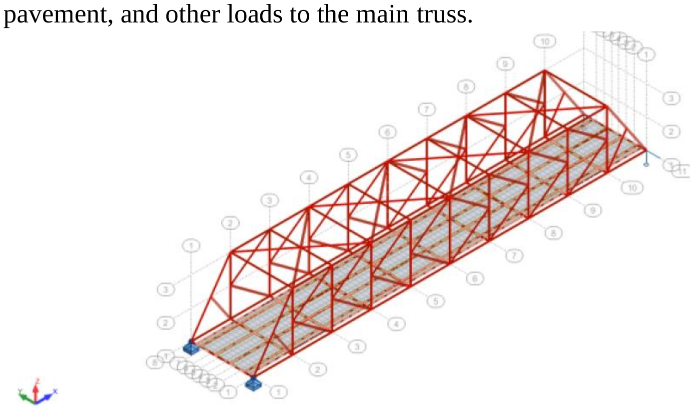
A. Patiraja bridge, the design method and its loading

The Patikraja Red Bridge was originally classified as a railway line during the Dutch era, but as time passed, it was repurposed as a public route passable by motorized vehicles. This bridge's construction has created complications due to the increased traffic flow and congestion in the Patikraja area, particularly near the Patikraja market crossroads, where cars must take turns crossing the bridge due to the bridge's short width.

Steel frame bridge remodelled utilizing type K with a width of 7 meters and a footbridge width of 0.5 meters, the bridge truss height is 6.3 meters, the bridge is constructed of steel components assembled with bolts and a backrest constructed from 75mm-diameter steel pipe. For the bridge's floor, floor concrete was used in conjunction with a wave plate deck. The bridge complies with highways standards and is classified as a permanent bridge class B.

The following is an overview of the technical specs for the 45-meter-long truss bridge using steel bridge structure K-type frame. The strength class and span respectively B-class and 45-m. (from elastomeric centers), while bottom chord length was 4.5 m and width of asphalt track width is 6 m; floor strip width including pedestrians on both sides as 7 m; left - right pedestrian road width was 0.5 m. The truss height = 6.3m measured from the scion center to the stem center lower.

The technique and design method can be describe as follows: First, calculating the load acting on the bridge in accordance with SNI 1725:2016 about Loading for Bridges. Second, the analysis of bridge trusses that is the primary structures designed to sustain the loads encountered on the bridge. Third, the wind ties or bracing purpose is to withstand or absorb the force generated by the wind, should be taken into account both at the top and bottom of the bridge. Fourth, the calculation of the transverse girders which is their purpose is to take and transfer the load of the vehicle's floor, pavement, and other loads to the main truss.



Node/Case	FX (kgf)	FY (kgf)	FZ (kgf)	MX (kgfm)	MY (kgfm)	MZ (kgfm)
1/ 1	-110,53	-14542,60	110805,66	0,00	0,00	-0,00
11/ 1	-0,00	-14533,40	110817,64	-0,00	-0,00	-0,00
31/ 1	110,53	14525,09	110817,64	-0,00	-0,00	-0,00
41/ 1	0,00	14550,91	110805,66	0,00	0,00	0,00
Case 1	MS					
Sum of val.	-0,00	-0,00	443246,60	-0,00	-0,00	-0,00
Sum of reac.	-0,00	-0,00	443246,60	1580174,13	-9973048,52	-0,00
Sum of forc.	0,0	0,00	-443246,60	-1580174,13	9973048,52	0,00
Check val.	-0,00	-0,00	-0,00	-0,00	0,00	-0,00
Precision	7,87355e-06	8,66704e-20				

Fig. 3 Geometry layout and selfweight deadload of Patiraja K-truss bridge

For the bridge fixed load, including the floor, is 443246.60 kg according to the findings of the analysis performed using the 2017 Robot Structural Analysis Program (RSAP). The pavement weight was a 25 cm design thickness (according to B-class permanent bridge classification), The concrete used was a low mass density of 1250 kg/m^3 and the asphalt pavement was 5 cm thick and has a density of 2245 kg/m^3.

The "D" lane loads are composed of uniformly distributed loads (BTR is equal to UDL) and concentrated line loads (BGT is equal to KEL) with an intensity of p kN/m that must be installed perpendicular to the bridge's traffic direction. The intensity p has a magnitude of 49.0 kN/m as illustrated in Fig. 4.

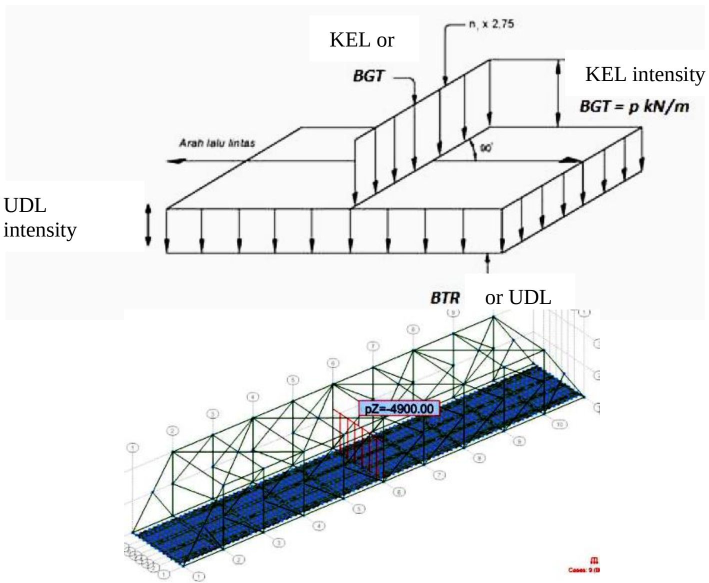


Fig. 4 Live load UDL and KEL in SNI 1729-2016 Loading for Bridges and their implementation on the bridge
Source: [13]

For the bridge traffic load, the D-live load, uniform deal load UDL was = 9(0,5+15/L) kPa = 7.5 kPa = 750 kg/m^2 according to Indonesian bridge code. While P-live load which is knife equivalent load, KEL was = 49 kN/m, and truck axle load, T was = 50kN ; 225kN; 225 kN such depicted in Fig. 5.

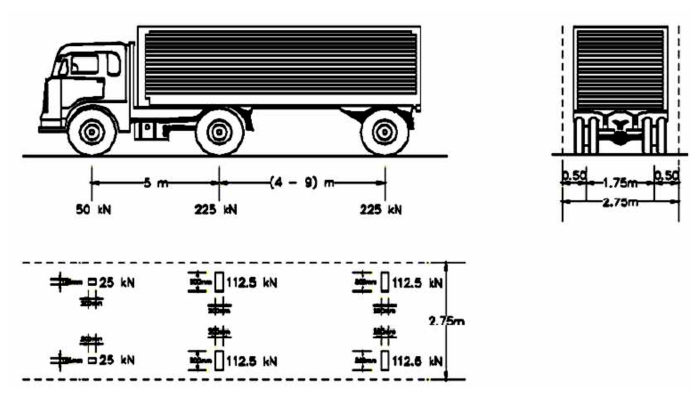


Fig. 5 Truck axle loads in SNI 1729-2016 Loading for Bridges
Source: [13]

The brake load was taken the largest value of two, 25% axle weight = 125 kN and other as 5% design truck weight + UDL = 1746.25 kN, so 1746.25 kN controlled as maximum value between both. Hence, the brake moment can be computed as M_{TR} = 1746.25 kN.(1.8) = 3098.25 kNm as 309825 kg.m. While the pedestrian load on the sidewalk in accordance with the loading code of SNI 1725:2016:46 is 5 kPa.

For the wind load, here are the assumption made, structural elevation (Z) as 19000 mm from the sea, and the friction length upstream of the bridge (Z_0) as 70, while the wind friction speed (V_0) as 13.2. The wind speed at elevation 10 m (V_{10}) as 100 km/hour. Base wind speed was calculated from 90-126 km/hour (V_B) as 90 km/hour was chosen. Using bridge point as 30 nodes, and base wind pressure and suction respectively $BWP_{pressure}$ as 0.0024 MPa and $BWP_{suction}$ as 0.0012 MPa.

Hence, wind speed design (V_{DZ}) can be computed as

$$V_{DZ} = 2.5V_0.\left(\frac{V_{10}}{V_B}\right) ln\left(\frac{Z}{Z_0}\right) = 205.468 \text{ km/hour}$$

From the data above, the design wind pressure (PD) is as follows,

$$P_{D-pressure} = P_B.\left(\frac{V_{DZ}}{V_B}\right)^2 = 0.0125 \text{ MPa}$$

Using the same manner the PD-$_{compression}$ value in the form of load as 0.0125 MPa equal 12500 Pa equal = 35437500 kg divided by 30 nodes results in 1181250 kg/nodal. Directly the PD-$_{suction}$ as 0.00625 MPa equal as = 6250 N/m^2 equal = 17718750 kg divided by 30 nodes results in 590625 kg/nodal. Equivalent wind load on vehicle, EWV = 672 kg.m and equivalent wind load in bridges, EWL as 6570 kg divided by 30 nodes =298,636 kg/nodal.

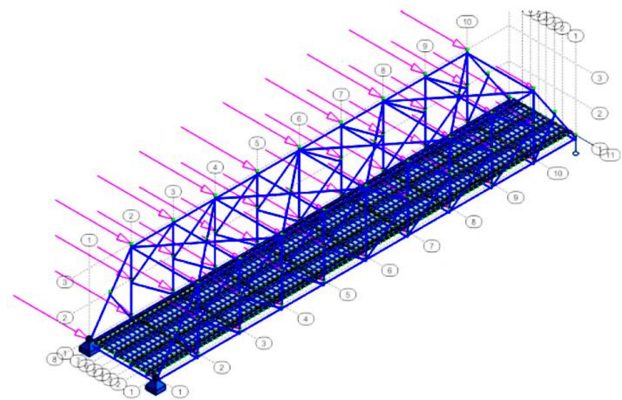


Fig. 6 Wind load implementation on the bridge

B. Analitical and design results

The largest load combination is chosen from the 12 load combinations entered in the RSAP, namely Strength-1, which consists of a combination of loads (1.1MS+ 2MA+1.8 UDL + 1.8 KEL + 1.8 TB+ 1.8 T) and the static analysis results in the maximum compressive force is 541225.14 kg, while the maximum tensile force is 364721.49 kg such as depicted in Fig. 7.

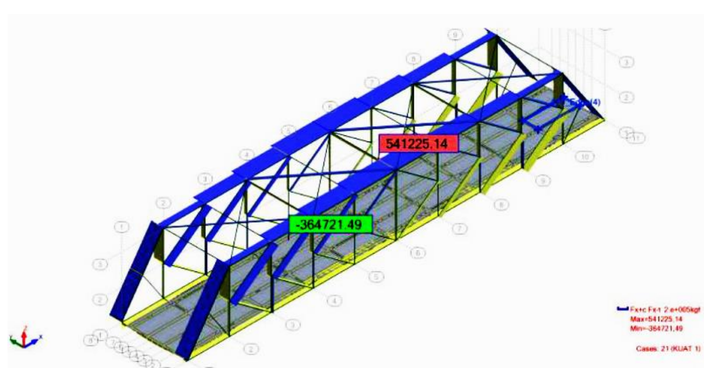


Fig. 7 The axial force (Fx) bridge trusses generated by STRENGTH-1 load combination

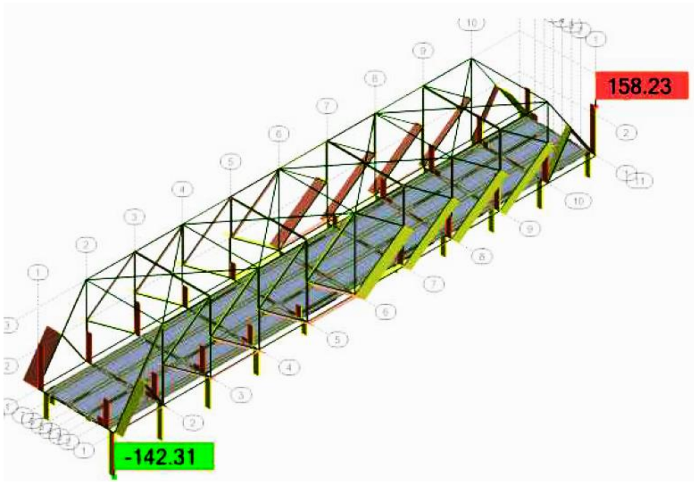


Fig. 8 The torsional moment (Mx) of end-girder generated by STRENGTH-1 load combination

The maximum torsional moment in the X axis direction is 158.23 kgm as described in Fig. 8. No shear force is occurred in the truss member and the maximum deflection is found as 36 mm as depicted in Fig.9 and maximum tension force of wind bracing was 61804.14 kg such illustrated in Fig. 10.

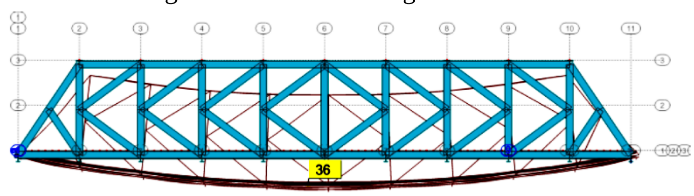


Fig. 9 The maximum deflection occurred in the mid-span of truss bridge generated by STRENGTH-1 load combination

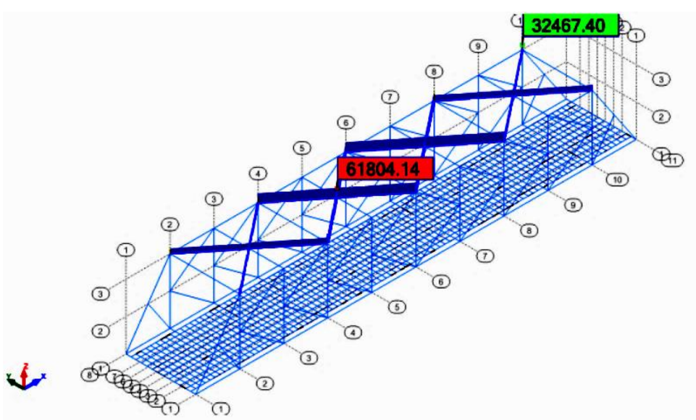


Fig. 10 The tension force occurred in the wind bracing generated by STRENGTH-1 load combination

The nominal tensile capacity of the End Wind Bracing (EWB), the Diagonal Truss Member (DTM), and the Bottom Truss Member (BTM) is calculated using welded section H-beam of 500.500.25.40 against the axial force tensile Pu = 364721.49 kg fully plastic capacity and fracture capacity due to holes existence in the tensile connection. The fracture of effective area described in Fig.11.

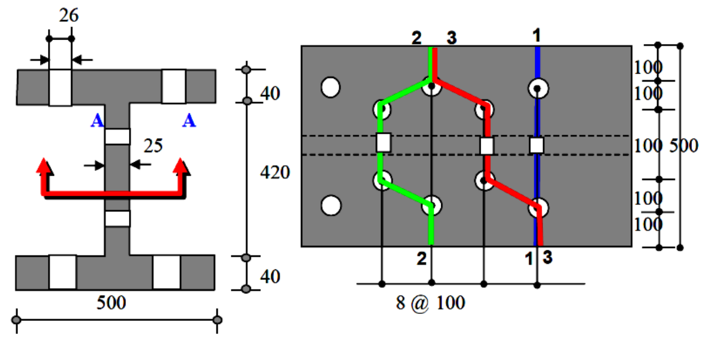


Fig. 11 The implementation of effective area fractured-risk taken into account on tension member

The tension member should be controlled under yield condition, $\phi T_{n1} = 0.9.A_g.F_y = 20907000$ N and fractured condition, $\phi T_{n2} = 0.75.A_e.F_u = 17706562{,}5$ N respectively. Since both slightly larger than Pu, hence the H 500.500.25.40 profile can be used as a design profile. The longitudinal girder and lateral main girder also examined under STRENGTH-1 load combination and their moment are depicted in Fig.12 and Fig. 13 respectively.

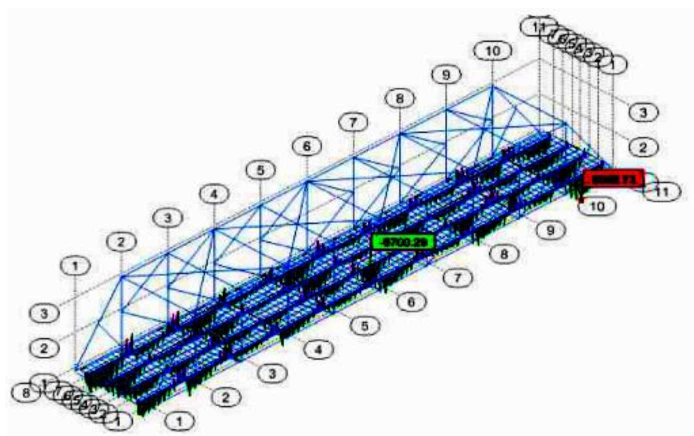


Fig. 12 The maximum moment occurred in the longitudinal girder generated by STRENGTH-1 load combination

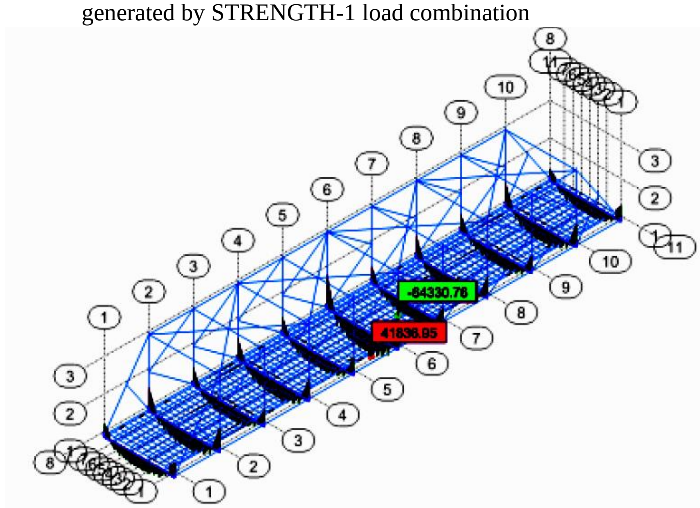


Fig. 13 The maximum moment occurred in the lateral main girder generated by STRENGTH-1 load combination

IV. CONCLUSION

The following conclusion can be drawn from the design of the Patikraja Red Bridge using a K-Truss Type and the following dimensions are determined for the steel profile required for the truss bridge:

a. The steel profile H 500.500.25.40 is utilized for the Top Truss Member (TTM), Diagonal Truss Member (DTM), and End Wind Bracing (EWB) and Bottom Truss Member (BTM).
b. The H-500.200.10.16 steel profile is used for the Wind Tie Member (WTM).
c. The H-700.300.13.24 steel profiles for transverse and end transverse girders (GMT & GMU).
d. Steel profile for longitudinal girder (LG) used was H-450.200.9.14
e. Steel profiles for Transverse and End Transverse girders (TG & ETG) used was H-700.300.13.24.

Basically, a K-Truss Type show a good results in internal forces and maximum deflection,

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