Structural Evaluation of Railway Warren Steel Truss Bridge with Span Length of 42 Meter According to SNI 2833:2016 and Earthquake Hazard Map 2017

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Abstract—**Railway bridge with the warren type truss structure in the Sumatera region is an infrastructure has existed since the Dutch colonial era. Bridge is used as a mode transportation for natural resources of coal. The construction of railway bridge must always be monitored and evaluated for the feasibility of the structure, especially for earthquake load. This is due to change the latest of earthquake hazard maps and regulations regarding earthquake load. The research used SNI 2833:2016 regulations, about bridge planning for earthquake load and 2017 earthquake hazard maps, by modelling the 42 meter warren type of truss structure using SAP2000 V.21 software. The results of the research indicate that, the largest of mode shape value occurs UZ point with the 6 mode number of 0,82536 displacement unit and a period value of 0,075276 second. The largest of period (T) value occurs in 1 mode number of 0,37002 second. The maximum displacement occurs U3 point at the joint 25 of 36,437377 mm opposites the axis. The value of displacement in the direction of U3 is smaller than the value of allowed deflection of 52,5 mm. Then, at the stress control, there are 13 frames that have overstressed and the maximum stressed of frame occurs on the pressure frame, the IWF150.150.7.10 frame has the Pu value of 22,462 tons with a nominal compressive capacity (Pn) of 12,612 tons. Based on the 3 reviews, it can be concluded that, the bridge structure requires structural reinforcement on cross girder 1 (IWF1100.400.16.28), 2(IWF1100.400.16.28) and 7(IWF1100.400.16.28) frames and on wind bracing 13 to 22 (IWF150.150.7.10) frames.**

Keywords—railway bridges, warren steel truss, structural evaluation, earthquake hazard map 2017, SNI 2833:2016

I. INTRODUCTION

Bridge is a structure connecting separate regions due to obstacles, for instance, valleys, rivers, highways and seas. Nowadays, bridges support mainly two forms of transportation modas, common road and railways. In Indonesia, railways have been built since pre-independence era to transport sugar plantations and productions from factories [1]. Thus, trains can pass through these separate areas using these steel bridges. Some of the structural components in this infrastructure haven't been renewed and is still used today. One old railroad bridge that is still frequently used is in Lampung region, Sumatera Island. This railway bridge is connecting two points within 42 meters span separated by a river.

Indonesia is also a country which has large potential due to seismic hazard since it is in Ring of Fire area and is passed by four active plate tectonics [2]. Recently, the Indonesian National Center for Earthquake Studies has updated the newest earthquake hazard map in 2017 [3,4]. The earthquake hazard map is updated considering the occurrence of large earthquake events in Indonesia so that the data are always enriched to obtain more valid parameter.

The old bridge that had been designed and is still performed until now has to be checked and assessed periodically to ensure the safety of all passengers and goods transported through it. The checking process should follow the current condition including the provision and the needs. Many previous studies have been done before to assess bridge through some approaches. Since the bridge is a structure passed by vehicles many times, the fatigue might be one of many parameters of failure factor. One study conducted the fatigue assessment on bridges, specifically for railway truss structure using probabilistic stress-life method [5]. This method is predicting the remaining life of ageing bridges. Moreover, this study comparing the final predicted fatigue live from the aspect of validity, significance and rationality. Other research also assessed the fatigue life, but the assessment focused on component from t-jointed connection in steel truss bridge [6]. The investigation is targeting the effective notch method by conducting experimental and finite element analysis approach comprehensively. The subject of fatigue test was prototype joints which usually used in welded joint of steel bridges. The conservative image of steel bridges strength can be known from the research.

Fatigue is one of many assessments that can be conducted to seek for the security of old bridges. However, many assessments also can be done by measuring all related

parameters directly in the field, for example using fiber optic distributed as a sensor to read the strain indicator [7] and even the detection of fault and monitoring can involve artificial intelligence (AI) with fuzzy-based network [8]. After the assessment, the future action from the result has to be considered as well. In one study, the assessment performed with restoration procedure respectively [9]. From the fatigue, the remining age could be predicted as well as this age could be prolonged by many method, one of which is using mass damper system [10].

Other important parameter that could damaging the bridge is seismic load. One study performed numerical analysis to see the seismic response of cable stayed bridge [11]. Steel truss bridges have been some subjects to other research in their study, such as proposing self-supporting method to replace element on truss bridge [12], conducting non-linear analysis to seek the mechanism of failure in truss bridges [13], and for long span simply supported truss bridge for railways [14].

Another study related to railway bridges to seismic activity is done in 2015. The study propose a retrofitting steel truss bridge structure caused by seismic using fluid viscous damper [15]. The safety analysis of high-speed railway on bridges also has been conducted due to seismic excitations [16]. Not only that, the interaction of welded connection is also examined from seismic activity [17]. Other test was also conducted to see the vulnerability of the railway bridge [18], while structure damage detection is also proposed by its dynamic characters in other study [19]. Many other research related to steel truss bridges under seismic namely in Turkey [20] and for pedestrian bridges [21]. Beside the seismic load, the moving load also became a factor that give the vibration or dynamic activity in the bridge [22].

From the many studies mentioned above, only few research of assessment use the displacement parameter, one of which is by performance-based assessment of railroad bridges under service traffic [23]. In this study, an old steel truss bridge structure for railways in Lampung region is assessed by comparing the displacement and the strength ratio to its limit based on the AISC 2010 or Indonesian national standard for steel structure SNI 1729:2015. The load that is used is based on the earthquake bridge provision SNI 2833:2016 supported by the newest earthquake hazard map 2017 and the normal load for bridge based on SNI 1725:2016. The demand and capacity comparison of displacement and strength for existing steel bridge elements are discussed in this paper.

II. RESEARCH SUBJECT AND METHOD

In this study, the bridge that became the object of analysis in this study was a railroad bridge in the operational area of Lampung region, Sumatera Island. The following is the technical data from the structural bridge dimension. Also, the illustrations of two and three dimensional bridge dimension are shown in **Error! Reference source not found.** and.**Error! Reference source not found.**.
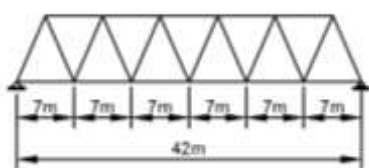

Fig. 1. 2D dimension bridge from side view

Fig. 2. 3D bridge illustration and bridge width

The material used in this study is steel which the material and dimension is based on ASTM A36 and has a yield stress of 36 ksi ($f_y = 250$ MPa, $f_u = 400$ MPa) with Young's modulus of steel (E_s) of 2.1 x 106 kg / cm^2. The modeling is performed using SAP2000 v21 which the license is provided in Computation Laboratory, Department of Civil Engineering, UMY Indonesia. These modellings follow the existing bridge design criteria, with a span of 42 meters and a bridge width of 4.9 meters. The procedures of this study are elaborated below.

a. Bridge designs are drawn in AutoCAD 2016 to find out the length of steel profile bars on the bridge structure.

b. Loading calculations is performed that to be inputted to the bridge structure modeling in the structural analysis program that refers to the following regulations:
   1) Indonesian Loading for bridges (SNI 1725: 2016)
   2) Indonesian PM No. 60 of 2012, Regarding the Technical Requirements of the Railroad
   3) Indonesian PD No. 10 Railroad Construction Planning
   4) Indonesian Bridge planning for earthquake loads (SNI 2833: 2016)
   5) Indonesian 2017 Earthquake Map

c. Bridge structures are modelled with structural analysis programs using SAP2000 V.21 software.

d. The type of steel profile is determined in each part of the bridge with SAP2000 V.21 software.

e. Loads that have been calculated based on structure model is inputted. These include dead load, live load, impact load, railroad lateral load, brake and traction load, centrifugal load, longitudinal rail load, wind load, earthquake load.

f. Live load model was formed in moving load from the train by using the moving load tools on SAP2000 V.21 software.

g. Loading combination that will be inputted on the bridge structure model was determined based on valid provision. The combination of loading is done to determine the maximum load that works from all the loads that have been inputted on the bridge structure model.

h. The bridge structure model that was made on SAP2000 V.21 software is checked. The important thing to check is the type and dimensions of the profile modeled and the loads that work on the bridge.

i. "Run Analysis" is performed to obtain the output and result of structural analysis from the bridge modeling that has been made.

III. RESULT AND DISCUSSION

A. Mode Shapes

The value of the maximum shapes mode whose dominant movement towards UX occurs in mode number 6 is 0.82536 displacement units with a period value of 0.075276 seconds. For the dominant movement towards UY occurs in mode number 1 which is 0.432378 displacement units with a period value of 0.370023 seconds and for the dominant movement in the direction of UZ occurs in mode number 4 which is 0.7308 units of displacement with a value of 0.124213 seconds. The biggest vibration period produced on the bridge structure has a period value (T) of 0.37002 seconds in mode 1. These values can be seen in Fig. 3 to Fig. 5.

Fig. 3. Period and mode shapes displacement in X direction

Fig. 4. Period and mode shapes displacement in Y direction

Fig. 5. Period and mode shapes displacement in Z direction

The mode shapes happened in the model can be illustrated as three-dimensional deflected steel truss bridge as showed in Fig. 6 to Fig. 8 below.

Fig. 6. mode shapes in X direction

Fig. 7. mode shapes in Y direction

Fig. 8. mode shapes in Z direction

B. Displacement

The maximum displacement value in the direction of U1 occurred at joint 51 (Fig. 9) with a value of 11.222773 mm, in the direction of U2 occurred at joint 54 (Fig. 10) with a value of -0.615116 mm and in the direction U3 occurred at joint 25 (Fig. 11) with a value of -36,437377 mm. The sign min (-) shows the direction of displacement opposite to the direction of the axes U2 and U3. In Fig. 12, Fig. 13 and Fig. 14 show a graph of the displacement values in each direction, namely U1, U2 and U3.
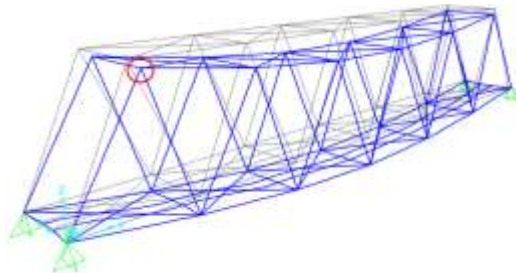

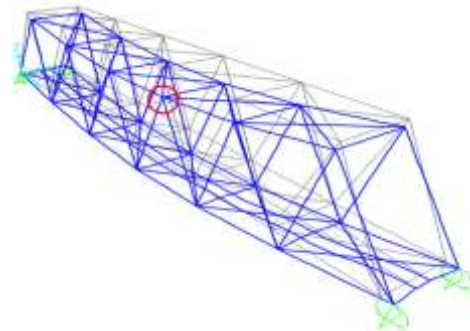Fig. 9. Period and mode shapes displacement in Z direction

Fig. 10. Period and mode shapes displacement in Z direction
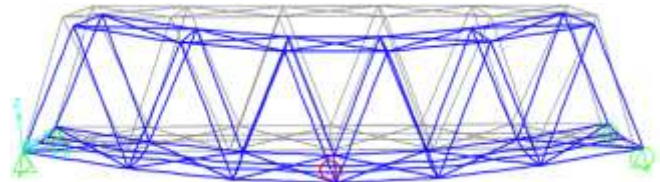

Fig. 11. Period and mode shapes displacement in Z direction
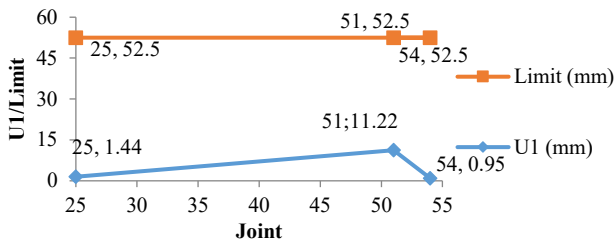
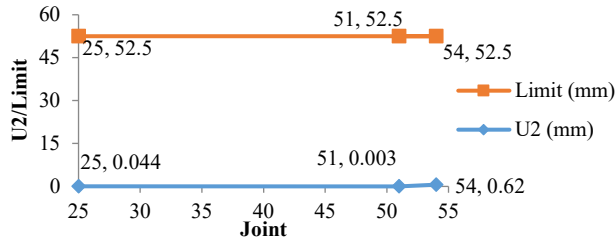Fig. 12. displacement in X direction
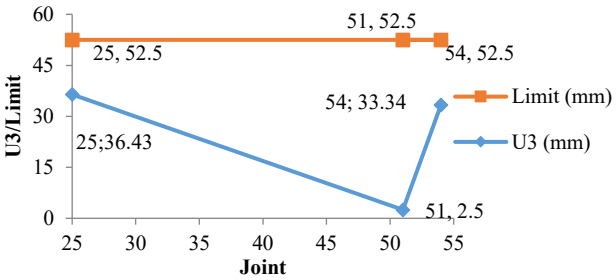

Fig. 13. displacement in Y direction

Fig. 14. displacement in Z direction

The displacement results that occur at U3 at the joint 25 have a value of 36.437377 mm opposite U3 smaller than the permit deflection value, which is equal to 52.5 mm. So, the structure is declared safe against deflection that occurs.

C. Stress in Elements

The stress parameter is controlled based on several applied combination is the running analysis which can be seen in Table I.

TABLE I. COMBINATION APPLIED IN RUNNING ANALYSIS

No.	Case Name	Information
1	DL	Safe
2	DL+ADL	Safe
3	DL+ADL+WIND	Safe
4	DL+ADL+ WIND +IMPACT	Safe
5	DL+ADL+ WIND +IMPACT+BREAK	Safe
6	DL+ADL+ WIND +IMPACT+BREAK LATERAL	Safe
7	DL+ADL+ WIND +IMPACT+BREAK LATERAL+LONGITUDINAL	Safe
8	DL+ADL+ WIND +IMPACT+BREAK LATERAL+LONGITUDINAL+TEMPERATURE	Safe
9	DL+ADL+ WIND +IMPACT+BREAK LATERAL+LONGITUDINAL+TEMPERATURE+EARTHQUAKE	Safe
10	DL+ADL+ WIND +IMPACT+BREAK LATERAL+LONGITUDINAL+TEMPERATURE+MOVING LOAD	Not Safe
11	DL+ADL+ WIND +IMPACT+BREAK LATERAL+LONGITUDINAL+TEMPERATURE+MOVING LOAD+EARTHQUAKE	Not Safe

The Run Analysis and check structure results in conditions 1-9, steel frames are safe for loads found in conditions 1-9 when running (Fig. 15). In conditions 10 and 11, the train load / moving load is added and carried out, both have the same capacity ratio value. The results show that there are 13 steel frames that have failed capacity or unsafe conditions (Fig. 16). The failed frame is the upper wind bond and 3 transverse girder.
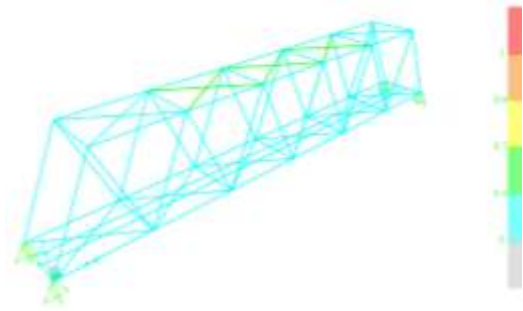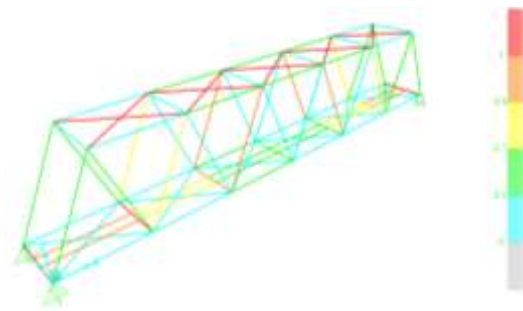

Fig. 15. Safe condition

Fig. 16. Ultimate condition

There are 3 frames that were overstressed on the transverse girder in the form of IWF1100.400.16.28 steel profiles. The ratio frame graph can be seen in Fig. 20 and the rod / frame position which is overstressed as seen in Fig. 17 to 19.
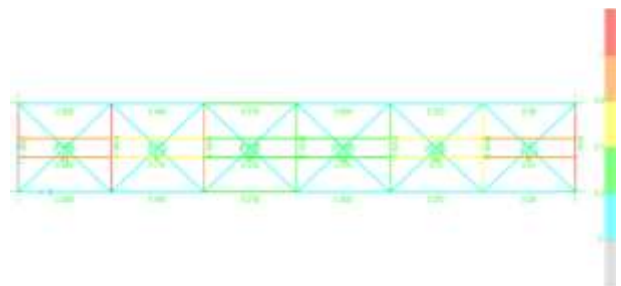

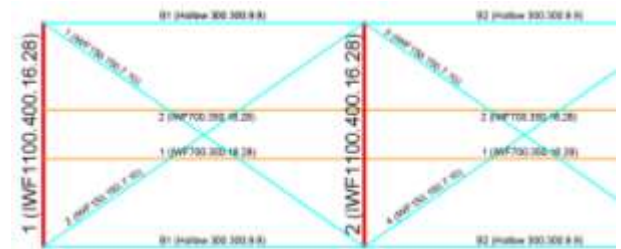Fig. 17. overstressed frames viewed from longitudinal view

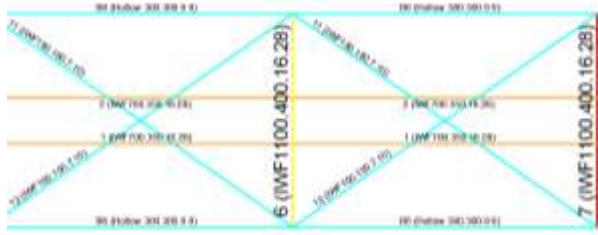Fig. 18. overstressed frame number 1 and 2 (IWF1100.400.16.28)

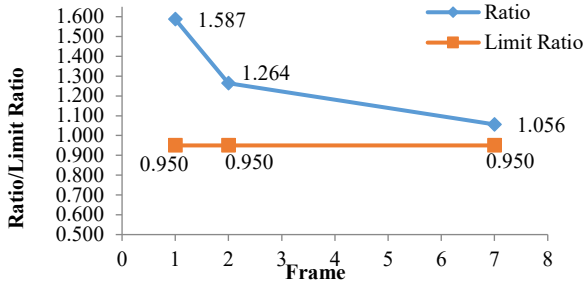Fig. 19. Overstressed frame number 7 (IWF1100.400.16.28)

Fig. 20. Ratio and limit ratio for frame number 1, 2, and 7 (IWF1100.400.16.28)
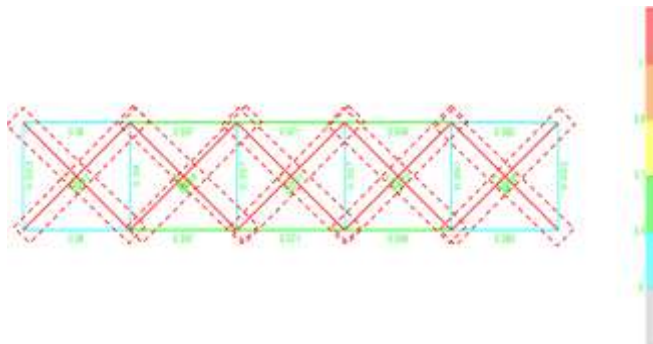

Fig. 21. overstressed happen in wind bracing

Fig. 22. Detailed of overstressed frame number 13-18 (IWF150.150.7.10)

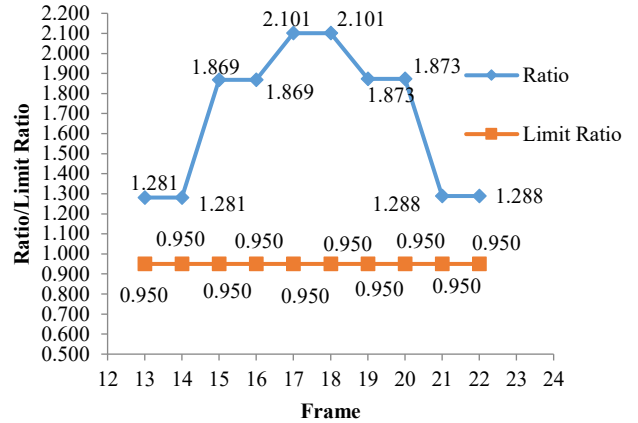Fig. 23. Detailed of overstressed frame number 19-22 (IWF150.150.7.10)

Fig. 24. Ratio and limit ratio for frame number 13-22 (IWF150.150.7.10)

The results of the magnitude of the resistance in the frame that was overstressed. Each frame has a voltage capacity capacity to be able to withstand the stress that occurs in the frame. the frame that experiences the greatest stress occurs in the press rod, namely frame 17 and 18 (IWF150.150.7.10) with a Pu value of 25,324 tons with a compressive capacity of 12,612 tons. Thus, the frame is declared insecure against the amount of compressive stress that occurs.

IV. CONCLUSION

To sum up, it can be concluded that he maximum shapes mode values of the three directions, namely UX, UY and UZ occur at the point with the dominant movement towards UZ with mode number 6 equals to 0.82536 displacement units and with a period value of 0.075276 seconds. For the largest period (T) value occurs at number 1 mode for 0.37002 seconds. The maximum displacement occurs in each direction. In the direction of U1 at joint 51 is 11.22 mm, U2 at joint 54 is -0.62 mm and U3 at joint 25 is -36.44 mm. Displacement behavior in the direction of U2 and U3 is opposite to the direction of the axes U2 and U3. For the displacement in the direction of U3 is deflection that occurs, with a value of 36.437377 mm smaller than allowable deflection, which is equal to 52.5 mm. So, the structure is declared safe. Structures is in field capacity condition in 13 elements when the earthquake load x and y and from moving load. This elements is in the transverse girder position, which is frame 1 (IWF1100.400.16.28), 2 (IWF1100.400.16.28), 7 (IWF1100.400.16.28) and the wind bracing, which are elements number 13 to 22 (IWF150.150.7.10) with the maximum capacity ratio values in elements number 17 and 18 (IWF150.150.7.10) of 2.101 which are overstressed. For the maximum compress value on frame 17 and 18 is 25,324 tons with a nominal capacity of 12,612 tons. This can be known that the stress value is not safe for the combination applied and must be reinforced. Moreover, this study is done numerically with the ideal and homogenous steel strength, thus the future research and assessment toward this bridge is suggested to consider the real condition and many other affecting parameters such as current damages, corrosions, and real leftover strength depicted by nominal yield stress by conducting several field tests.

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