JCE

Journal of Civil Engineering
ISSN 2086 - 1206 E-ISSN 2579 - 9029

Effect of the Increase in Vertical Web Member Stiffness on Lateral Buckling Strength of the Pony Steel Bridge

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Submitted : 13 February 2025
Revised : 19 February 2025
Accepted : 20 February 2025

Abstract

In half-through bridge or pony steel bridge, that is a bridge without upper wind bracing, strength of the bridge is determined mainly by the lateral buckling strength of its upper chord. Buckling strength of this chord is provided by the flexural stiffness of vertical web member, cross beam, and diagonal beam. In order to improve the stiffness of vertical web member, triangular steel profile that was quite high was added to the inner side of bridge for reducing the clearance width in bridge and disturbing traffic or pedestrian. In this research, stiffness of the vertical web member was improved by using the non-prismatic cross section and adding the triangular stiffener as high as the concrete deck. Finite Element Analysis for the lateral stiffness of bridge cross section used a 3D element model which has been validated by previous study. This numerical study was conducted to validate the Engesser theory for determining the lateral elastic stiffness from upper chord. Study shows the result that accuracy of 3D element model is extremely high, compared with analytic method. Lateral elastic stiffness of bridge in general increased along with the stiffness of vertical web member. However, it can be concluded that effect on the capability of lateral buckling in upper chord was not too significant, as a consequence of the increase in stiffness of vertical web member. Critical lateral buckling occurred in an inelastic range, in which the critical inelastic buckling stress was determined using small tangent modulus as alternative of modulus of elasticity.

Keywords

Ponny bridge, lateral buckling, lateral stiffness, inelastic range, 3D element

INTRODUCTION

Strength of the pony steel bridge as the consequence of gravitational load is determined by the capability of lateral buckling in upper chord. This chord, having no lateral bracing, can buckle with length of the lateral buckling exceeding that of panel length or distance between two nodal points. Lateral buckling length is affected by the virtual elastic spring stiffness, that is the contribution of flexural stiffness from vertical web member, cross beam, and diagonal beam. Analytic study has been frequently conducted by Engesser since 19th century and continued by other researchers, [1-4]. By modelling upper chord as compression element supported by the elastic spring along its lateral buckling, critical buckling strength is determined. Assumptions on upper chords from the analysis in [1]: prismatic chords in which their tips are pin, lateral chords are concentrated elastic with the same stiffness and compressive force in constant chords. Approach analysis using the energy method, for lateral stability from plane truss by considering flexure and torsion, was investigated by [5]. Analytic study and FEM, with respect of the study in [1-4], were investigated further by [6]. Study shows the result that lateral spring stiffness calculated with analytic and FEM methods had no meaningful difference. Experiment for lateral buckling in steel frame with hollow section profile and flexible lateral bracing was conducted by [7]. Study shows the result that lateral buckling length was below the recommendation in Eurocode 3. Another study from [8] was conducted in pony steel frame made of aluminium. The use of trapezoidal model in its axial force resulted in buckling length that was similar between FEM and analytic.

This study aims to analyze the extent to which effect of the increase in vertical web member stiffness to the increase in critical compressive force from upper chord of a pony steel bridge. Result of the initial study, FEM model, that has been validated from [9,10] was used for numerical analysis for determining the lateral stiffness from upper chords. In the designing process, this study used the method of Allowable Strength Design (ASD); when upper chords are loaded with force in such a way, the occurring tension is above its proportional limit or belongs to the inelastic region. In the inelastic zone, value of the critical stress is tangent modulus function that is stress function to the slenderness [11,12]. For practical use, tangential modulus can be calculated with a method in [13,14].

MATERIALS AND METHOD

The first step in this study is to validate the finite element model from previous study that will be used for determining lateral stiffness of upper chord in pony bridge. Result of the model that has been validated was used to calculate the lateral stiffness from several cross-section models for bridge that has been modified with additional stiffener on its vertical web member. Based on the lateral stiffness that has been acquired, critical lateral buckling from upper chord can be determined. In the design, critical compressive force from upper chord belongs to the inelastic range, while buckling length value needs to be modified with tangential modulus instead of elasticity modulus. To determine the tangential modulus, an iterative procedure was used until the allowable inelastic stress converged [13].

1. Lateral stiffness validated using FEM

In the analysis to calculate the lateral elastic stiffness, C, FEM support program was used to model the panel which consisted of vertical web member and cross beam as the three-dimensional element network 3D and wire model. Model assumed to be fixed in the centre of cross beam was loaded in horizontal direction on peak of vertical web member to obtain lateral displacement for one unit. Accuracy of the element model used was validated with result of previous study from [6] and analytic study with Engesser formula [1] with the Equation (1).

$$C = \frac{E}{h^2\left[\left({h}/{3I_c}\right)+\left({b}/{2I_b}\right)\right]} \quad (1)$$

where: E= modulus of elasticity; h= height of truss; b= length of cross beam; I_c= moment of inertia in vertical web member; I_b= moment of inertia in cross beam

When contribution of diagonal web member with length L_d and moment of inertia I_d were calculated by adding stiffness, Equation (1) was modified into Equation (2). From this Equation, C value was larger by including the stiffness of diagonal web member.

$$C = \frac{E}{\left[\left(\frac{h^3}{3I_c + 3I_d\left(\frac{h}{L_d}\right)^3}\right)+\left(\frac{h^2 b}{2I_b}\right)\right]} \quad (2)$$

2. Lateral inelastic buckling of upper chord

Lateral stiffness of vertical web member C obtained from Equation (1) can be used to calculate the length of lateral buckling, v, and critical buckling force, Nc, from upper chord with Equations (3) and (4) respectively. Tangential modulus in Equations (3) and (4) can be determined with Equation (5).

$$v = \pi\sqrt[4]{\frac{E_t I_c L}{C}} \quad (3)$$

$$N_c = 2\sqrt{\frac{C E_t I_c}{L}} \quad (4)$$

where: Et = tangential modulus; L = length of panel

Tangential modulus from Equations (3) and (4) was obtained with iteration method from Equations (5) and (6), so the allowable inelastic stress, F_a, was convergent. This procedure was similar with what was conducted by [13] to determine the inelastic column buckling. Another method was to determine the upper limit F_a by $0.6F_y$ [14]. From Equation (6), α is a ratio of the allowable inelastic stress to Euler buckling stress, F_e'. F_a was determined using formula in the AISC-ASD [15] regarding Figure 1. To calculate the safety factor in upper chord, Nc obtained from Equation (4) was divided by the compressive force as a result of live and dead loads in service condition, S_a, as shown in Equation (7).

$$E_t = \alpha E \quad (5)$$

$$\alpha = \frac{F_a}{F_e'} \quad (6)$$

$$SF = \frac{N_c}{S_a} \quad (7)$$

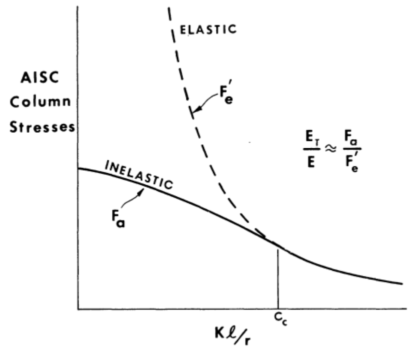


Figure 1 AISC column stresses [13]

3. Implementation on the bridge model

Procedure above was applied on the bridge model with span of 18,400, bridge width of 7,560 mm, and panel length of 1,840 mm as shown in Figure 2. Live load on bridge had 9 kN/m^2 of Uniform Distributed Load (UDL) and 49 kN/m of Knife Edge Load (KEL), with Dynamic Load Allowance by 1.4 based on Indonesian loading standards for bridges, SNI 1725 [16]. Besides UDL and KEL loads, based on AASHTO [17] for pony bridge, it needs to add lateral equal load on upper chords by 52.5 kN/m (0.3 klf). Cross section material in web member used Bj41 as provided in SNI 1729 [18] equal to A36-AISC. Cross section design in web member used ASD method. To analyze effectiveness of additional stiffener in vertical web member, model of transverse piece was made various: M-ori, M-250i, and M-908 as shown in Figure 3a, b, and c respectively. M-ori, M-250i, and M-908 were models without stiffener, inner stiffener with height of 250 mm, and outer stiffener with height of 908 mm (inner stiffener was added with a height of 250 mm), respectively. M-1108, M1308, M-1508, and M-1708 models were modification of M-908 models with outer stiffener height of 1,108 mm, 1,308 mm, 1,508 mm, and 1,708 mm as shown in Table 1. Stiffeners were made from cut sections of the vertical web member.

Table 1 Models with various stiffener's height

MODEL	NOTE
M-ori	Original, no stiffener
M-250i	Inner stiffener only; h=250 mm
M-908	Inner stiffener; h=250 mm + outer stiffener; h=908 mm
M-1108	Inner stiffener; h=250 mm + outer stiffener; h=1108 mm
M-1308	Inner stiffener; h=250 mm + outer stiffener; h=1308 mm
M-1508	Inner stiffener; h=250 mm + outer stiffener; h=1508 mm
M-1708	Inner stiffener; h=250 mm + outer stiffener; h=1708 mm

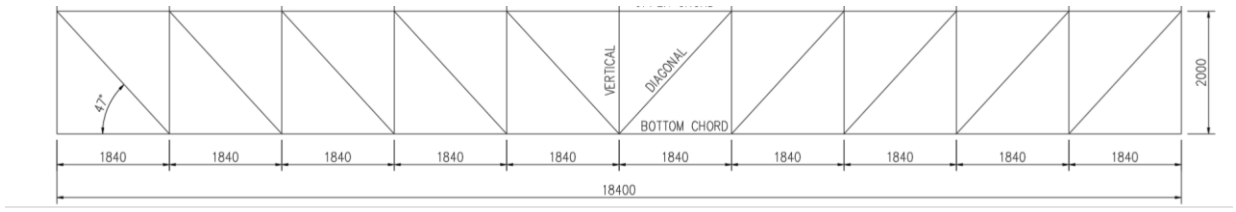
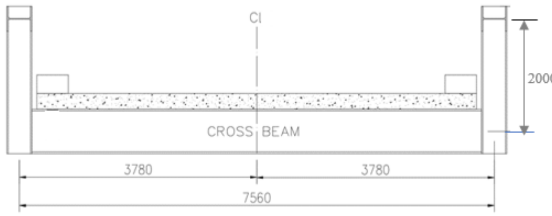
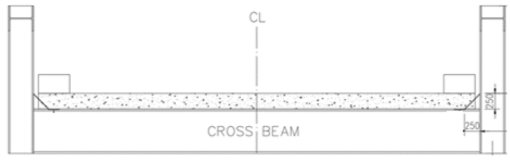


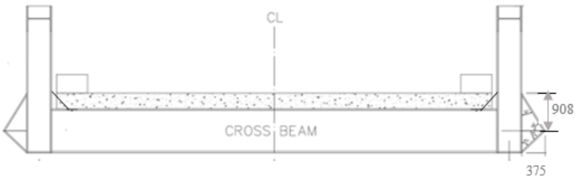
Figure 2 Displayed signal timing (Phases)



a) M-ori



b) M-250i



c) M-950

Figure 3 Cross-sections of the bridge with various heights of the vertical web stiffeners [9]

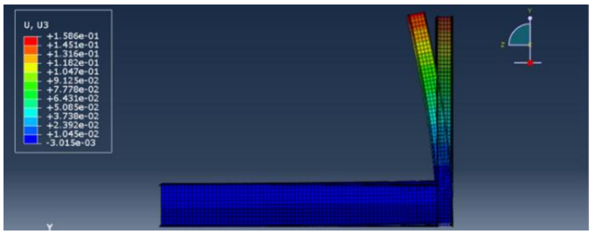


Figure 3 Cross-sections of the bridge with various heights of the vertical web stiffeners [9]

RESULTS AND DISCUSSIONS

1. Comparison of the lateral elastic stiffness C

From previous study [6] in which W27×84 with length of 8.897 m in was used for cross girder, vertical web member was used W10 x 33 with a height of 3.048 m and E = 200,000 MPa, by ignoring diagonal web member stiffness, C = 1152 kN/m (6.576 k/in) was obtained. This C value was verified with 3D FEM. Result of C comparison from model in Figure 4, wire elements [9,10], previous study [6] and the analytical method using Equation (1) were shown in Table 2. In 3D FEM, the value of C= 1158 kN/m was obtained from 1/Δ, where Δ= 0.0008636 m. Such a value corresponds to the displacement as a result of lateral force of 1 kN on the tip of vertical web member, as shown in Figure 4. Figure 4 shows the deformation of the M-ori model in kip-in units to validate the study [4] which still uses non-SI units. Table 3 shows the modelling using 3D element with high accuracy, with difference only 0.03% on the Engesser theory, so 3D element model can be used for the next analysis.

Table 2 Comparison of the elastic stiffness coefficient C using various methods

C (kN/m)			
FEM 3D	FEM wire element	Matthies [6]	Analytical Equation (2)
1158	1132	1152	1158

2. Cross section des ign of bridge element

Model Figure 1 was analyzed with the assumption that all joints in truss was rigid (fix). ASD method was used to obtain element cross section and minimum safety factor Ω, as ratio of nominal compressive strength to the required compressive strength (Ω= Sn/Sa) [15], as shown in Table 4. What needs to be observed is that upper chord was placed with strong axis (X-X axis) as its lateral buckling axis. Thus, Ω in Table 3 is buckling safety factor to the weak axis (Y-Y axis) with panel distance of 1,840 mm as buckling length. The observed upper chord was on the centre of bridge that resulted in maximum Sa force.

Table 3 Cross-section of the bridge member and minimum safety factor

Member	Section	Ω=Sn/Sa
Chords	WF 400×400×13×21	1.228
Vertical web	WF 400×400×13×21	1.325
Diagonal	WF 346×174×6×9	1.098
Cross beam	WF 708×302×15×28	1.101

3. Analysis of lateral buckling force and safety factor

Table 4 is an example to calculate Nc and SF for M-ori model with iteration method. Step-1: with C=17,113 kN/mm, result of the numeric analysis from Figure 3a, initial lateral buckling length was v=6,080 calculated with Equation (3) using elasticity modulus E=200,000 MPa. Fa= 136.0 MPa in column (4) and Fe'=852.4 MPa in column (5) obtained using formula AISC [15]. In this case, α=0.160 in column (6) can be calculated with Equation (6). Tangential modulus Et=31,926.8 MPa in column (7) was calculated with Equation (5). With Et obtained, critical buckling force Nc was calculated with Equation (4). Step-2: With a new Et, new v and Fa were 3843.0 mm and 142.2 MPa respectively. Iteration was continued until Step-7 in which Fa has been convergent (Fa=145.3≈145.2 MPa)

With the same method, elastic stiffness C from all numeric models was calculated and the result was in column (1) of Table 5. For example, the deformed models, M-250i and M-980, were shown in Figures 5 and 6 respectively. From C value, Nc and SF were calculated with iteration method and the result can be shown in column (9) and (10) respectively. Column (1) Table 5 shows that lateral stiffness C increased along with stiffener addition in vertical web member. When stiffener gets higher, C value is also higher. It is also followed with the decrease of buckling length v as shown in column (3). However, adding stiffener in vertical web member only increased the critical buckling force Nc and safety factor that was the same and insignificant, only 1.11%. It can occur when observing Equation (4) and the chart of Figure 1; although lateral elastic stiffness got higher, tangential modulus got lower along with the decrease of slenderness value λ, that made Nc relatively constant. From column (10) and Sa of 2,560.7 kN, without considering buckling on weak axis, safety factor from 4.700 to 4.750 was obtained. This design is still safe and meets ASSHTO [17] that requires safety factor of two. From M-250i, a model with inner stiffener only and height of 250 mm (on par with bridge deck) and M-1708, a model with additional outer stiffener and height of 1,708 mm, SF values of 4.729 and 4.750 mm were obtained. It suggests that additional outer stiffener was less effective because only adding safety by 0.44 percent. Furthermore, when using maximum assumption Fa by 0.6 Fy [14], maximum safety factor of 4.86 was obtained. This value was slightly larger when using method as recommended in [13].

Length of the inelastic lateral buckling from models that have been analyzed with iteration procedure (column 2 from Table 5) is shown in Figure 7b. This Figure only shows lateral buckling phenomenon along the bridge span for various stiffness in vertical web members, in which amplitude is not interest. This figure shows that buckling length is lower when vertical web member gets stiffer. Comparison of elastic buckling length before iteration analysis (step-1) from formula Equation 3 by replacing Et with E, is shown in Figure 7a. Similar to Figure 7b, this buckling phenomenon shows that lateral buckling length is lower when vertical web member gets stiffer.

Table 4 Nc and SF for the M-ori model using iteration method

Iteration step	v=KL (mm) (1)	λ=KL/r (2)	K (3)	Fa (MPa) (4)	Fe' (MPa) (5)	α=Fa/Fe' (6)	Et=αE (MPa) (7)	Nc (kN) (8)	SF (9)
1	6080.0	34.74	3.30	136.0	852.4	0.160	31926.8	27862.7	10.663
2	3843.0	21.96	2.09	142.2	2133.4	0.067	13330.8	18004.2	6.890
3	3089.0	17.65	1.68	144.0	3302.0	0.044	8723.0	14563.9	5.574
4	2778.0	15.88	1.51	144.7	4081.6	0.035	7091.7	13131.7	5.026
5	2638.1	15.07	1.43	145.0	4527.6	0.032	6406.9	12481.6	4.777
6	2572.1	14.70	1.40	145.2	4763.0	0.030	6079.8	12175.5	4.660
7	2540.4	14.52	1.38	145.3	4882.2	0.030	5950.4	12028.7	4.603

Table 5 C, Nc and SF for various models

Model	C (N/mm) (1)	v=KL (mm) (2)	λ=KL/r (3)	K (4)	Fa (MPa) (5)	Fe' (MPa) (6)	α=Fa/Fe' (7)	Et=αE (MPa) (8)	Nc (kN) (9)	SF (10)
M-ori	17113	2540.4	14.52	1.38	145.3	4882.2	0.030	5950.4	12028.7	4.603
M-250i	23382	2182.0	12.47	1.19	146.0	6617.7	0.022	4413.1	12108.7	4.729
M-908	24960	2098.8	12.07	1.14	146.2	7057.0	0.021	4142.5	12121.0	4.733
M-1108	26104	2068.1	11.82	1.12	146.3	7367.5	0.020	3970.5	12135.5	4.739
M-1308	27613	2011.8	11.50	1.09	146.4	7783.3	0.019	3761.4	12148.2	4.744
M-1508	29082	1962.7	11.22	1.07	146.5	8176.7	0.018	3582.8	12167.7	4.752
M-1708	30297	1908.3	10.98	1.06	146.6	8529.3	0.017	3436.7	12163.4	4.750

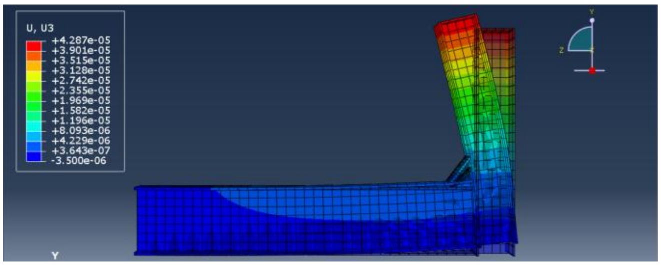


Figure 5 Deformed model of M-250i

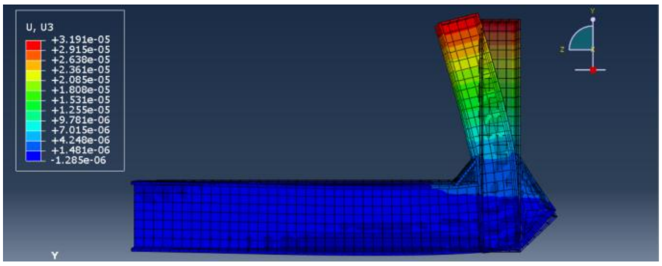
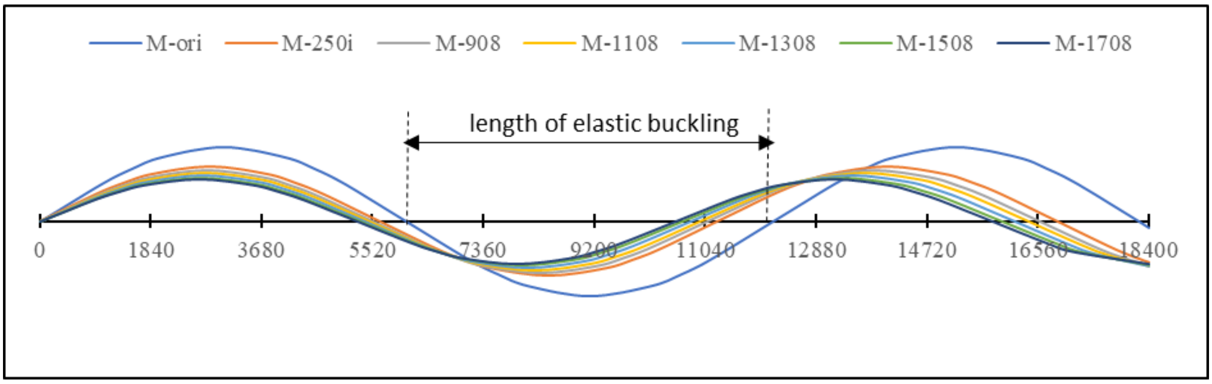
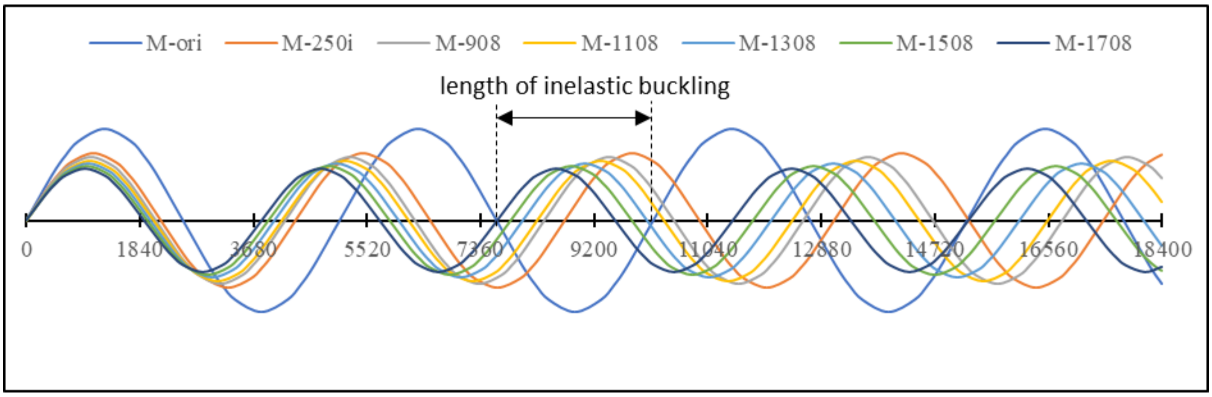


Figure 6 Deformed model of M-980



a. Elastic buckling



b. Inelastic buckling

Figure 7 Lateral buckling length of the upper chord for various models

Table 5 C, Nc and SF for various models

CONCLUSIONS

This numerical study was conducted to validate the Engesser theory for determining the lateral elastic stiffness of the upper chord in Pony Steel Bridge. Based on the obtained results, some conclusions drawn are as following:

1. In the use of 3D element model for calculating the lateral elastic stiffness, its validation on analytic and another FEM method is excellent.
2. Adding stiffener in the inner side and combination with outer side in vertical web member will improve the lateral elastic stiffness quite significantly.
3. The increase of lateral elastic stiffness from vertical web member will improve the lateral buckling strength in upper chord and its safety factor, though not significant. Lateral inelastic critical buckling from upper chord depends on tangential modulus that will shrink along with the decrease of slenderness value.
4. In the implementation for increasing inelastic lateral buckling strength from upper chord, stiffener as high as bridge deck needs to be added to the inner side from vertical web member.

ACKNOWLEDGMENTS

The authors would like to thank the Institut Teknologi Sepuluh Nopember for the support provided through the Research Grand No. 1952/PKS/ITS/2021

REFERENCES

[1] R. Malia, N. Fadhly and S. Sugiarto, “Traffic management of intersection with more than four road segments,” 4th Annual Applied Science and Engineering Conference (AASEC 2019), Journal of Physics: Conference Series vol. 1402, 2019, p. 022021.

[2] R. D. Ziemian, “Guide to Stability Design Criteria for Metal Structures,” 6th Ed. John Wiley & Sons, New York, 2010.

[3] E. Holt, “Buckling of a Pony Truss Bridge. Stability of Bridge Chords without Lateral Bracing,” Column Res. Council Rep. No. 2, 1952.

[4] F. Bleich, “Buckling Strength of Metal Structures,” McGraw-Hill Book Company, Inc., New York, 1952.

[5] S. P. Timoshenko and J. M. Gere, “Theory of Elastic Stability,” 2nd Ed. McGraw-Hill, New York, 1961.

[6] M. R. Horne, “The Elastic Lateral Stability of Trusses,” The Structural Engineer, pp. 147-155, 1960.

[7] D. Matthies, “Lateral Buckling Analysis of a Steel Pony Truss,” Theses, Ames IA Iowa State University of Science and Technology, 2012.

[8] J. Jankowska-Sandberg and J. Kołodziej, “Experimental study of steel truss lateral–torsional buckling,” Engineering Structures, 46(46), pp. 165-172, 2013.

[9] Q. J. Wen and Z. X. Yue, “Elastic buckling property of the upper chords in aluminum half-through truss bridges,” Structures, 27(27), pp. 1919-2929, 2020.

[10] H. Sugihardjo and A. B. Habieb, “Innovation of Half-through Bridge Cross Section to Improve Lateral

Stability and User Accessibility," Institut Teknologi Sepuluh Nopember Indonesia, Research Grand No. 1952/PKS/ITS/2021(in Indonesian), 2021.

[11] M. F. Rosyidi, H. Sugihardjo, B. Suswanto, and A. B. Habieb, "Comparison of Numerical and Analytical Method of Lateral Elastic Stiffness in Half-through Bridges Using FEM 3D," Jurnal Aplikasi Teknik Sipil Vol. 20, No. 1, pp. 95-100 (in Indonesian), 2022.

[12] T. V. Galambos and A. E. Surovek, "Structural Stability of Steel: Concepts and Applications for Structural Engineers," John Wiley & Sons, Inc., New Jersey, 2008.

[13] C. Salmon, J. Johnson, and F. Malhas, "Steel Structures Design and Behavior," 5th Ed. Pearson Prentice Hall, New Jersey, 2009.

[14] J. A. Yura, "The Effective Length of Columns in Unbraced Frames," AISC Engineering Journal, April, pp. 37-42, 1971.

[15] R. O. Disque, "Inelastic K-factor for Column Design," Engineering Journal, American Institute of Steel Construction, Second Quarter, pp. 33-35, 1973.

[16] American Institute of Steel Construction (AISC), "Steel Construction Manual," American Institute of Steel Construction Inc., 2011.

[17] SNI 1725, "Indonesian loading standards for bridges SNI 1725," Department of Public Works and Housings, Indonesia (in Indonesian), 2016.

[18] American Association of State Highway and Transportation Official (AASHTO), "AASHTO LRFD Bridge Design Specifications," 8th Ed., Washington, D.C., 2017.

[19] SNI 1729, 'Specifications for structural steel buildings," Department of Public Works and Housings, Indonesia (in Indonesian), 2020