VERIFICATION STUDY ON RESPONSE ANALYSIS OF TRUSS BRIDGE COMPRESSION MEMBERS

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ABSTRACT

Finite element structural analysis of truss bridge model with 2-direction earthquake motion has more significant effect in decreasing the strutural capacity compared to 1-direction motion. The present paper focusses on the significancy investigation of 1-direction and 2-direction earthquake motion to the maximum stress and strain of the compression members. Finite element model of a full truss bridge system was employed. The numerical results were verified using some utimate stress and strain formulae. The result verified that the 2-direction earthquake motion has much more significant effect in decreasing the bridge capacity and the escalation of maximum stress and strain establish the non-linear lines with the ground motion magnification factor. The most severe buckling occured in the members located near the bearing support.

Keywords : compression member, earthquake motion, respon analysis, truss bridge

1. INTRODUCTION

Few studies have been conducted on the ultimate strength of steel bridges during seismic activities compared to those on concrete bridges. Despite better ductility and resistance against earthquake, the structural strength and ductility of steel structures are subject to cross section design factors such as cross section type, *b/t* ratio, stiffening rib, as well as loading conditions. The ultimate state cannot be determined by the mechanical properties of the material only. The local buckling in steel plates must also be considered[4)].

Coupling two horizontal earthquake components have great influence on the response of bridge piers than when they are considered independently[1)]. However, it was neglected from conventional method of dynamic analysis assuming that self-weight of the structure can provide sufficient inertia to resist vertical motions.

The maximum average compression strains observed in 2D and 3D earthquakes have shown great similarity and their amplitudes have indicated close relationship with the fundamental natural period. Neglecting the exceptional cases, it can be said that 3 dimension earthquake has not significant impact on maximum strains in comparison with 2 dimension earthquake motion[2)].

Present paper deals with the dynamic analysis of a full truss bridge model. This model was assembled according to an actual railway truss bridge structure in Japan including bridge dimension and cross section properties of steel members. Actually, material property of real bridge structure is SS400 steel grade. But since the present study proposed formulations are suitable for SM490 steel grade, hence present truss bridge model was constructed in SM490 material property.

This analysis is aimed to verify the previous proposed equations[5)], including ultimate stress and strain. Most bridge structures in based design purposes are assembled using the conventional beam elements. That is why present chapter used beam type to verify the proposed equations in order to illustrate the actual bridge design procedure. Moreover, the previous chapters have proven that the use of beam models resulting good accuracy as well as shell model results. Present chapter is also aimed to investigate the influence of ground motion magnification in one and two direction of earthquake motion to the truss bridge compression members strength. As mentioned in the previous paragraph that 3 dimension earthquake has not significant influence on maximum strains compared to 2 dimension earthquake motion, present chapter used 1-dimension compared to 2-dimension earthquake motion in longitudinal and transversal bridge axis. Since most of bridge structure are designed to be able to carry severe earthquake loads, hence present chapter magnified ground motion data both in one direction and two direction earthquake motions.

2. NUMERICAL METHODS

Present paper analyzed full truss bridge model using the response method. All members in the present truss bridge FE model were assembled using B31 Timoshenko beam in ABAQUS software as the engineers are usually used in the practical fields. Large meshing pattern was used to simplify the running time process. Each span of a main superstructure was only divided into 5 to 20 segments depend on the analysis requirement. The main part that was used to verify the buckling phenomenon have smaller mashing patterns. Dynamic load with the implicit integration operators in ABAQUS/Standard was employed to evaluate the buckling loads. The boundary conditions used in present bridge systems were simplified in such a way that they can provide the bearing supports between superstructure and abutments. Simple pin in one sides and roller in another sides boundary condition was used. All members were rigidly connected. All types of initial imperfections such as initial displacement and residual stress were disregarded. Multi-point constrains were employed to impose constraints between members. Geometrical properties of main steel members of this truss bridge model is shown in **Table 1** where B is total width, D is the total depth and t is thickness. The layout of a full truss bridge model in the ABAQUS software can be seen in the **Figure 1.**

The present truss bridge model was assembled according to an actual dimensions of railway truss bridge in Japan. SM490 steel grade which has modulus elasticity (E) = 2 x 10^5 MPa, yield stress (σ_y) = 315 MPa and Poisson's ratio (υ) = 0.3 were used in the present analysis. Stress-strain curve input data was drawn as a bi-linear stress strain relationship with modulus elasticity slope in the strain hardening regime defined as E/100 (**Figure 2**).

Train load was necessary to be taken into consideration. Distributed gravity load and train load (TL) as 35 kN/m along bridge axis direction was employed. In the superstructure, train load was modeled as point mass (PM) which has total point mass as TL x L where L is total the length of truss bridge. The point masses were located in 8 certain location according to the bridge design requirement (**Figure 1**). Therefore, each point mass has the load as $PM = m$ x g where m is load applied in the point mass and g is gravity acceleration. All point masses have similar magnitude except the both edge sides which each edge point mass has half times of center point mass.

Table 1. Geometrical properties of main steel members

Bridge Part	Section Type	Dimensions (mm)
Main girder,main truss and overside beam	Box section	B = 320 D = 400 t = 15 R_R = 0.4242
Under side beam	I section	$B_{upper\ flange}$ = 300 $B_{lower\ flange}$ = 350 D = 843 t_{web} = 12 $t_{upper\ flange}$ = 21 $t_{lower\ flange}$ = 15 R_{flange} = 0.243 R_{web} = 1.466
Undercross and overcross beam	I section	$B_{flanges}$ = 200 D = 220 t_{web} = 8 $t_{flanges}$ = 10 R_{flange} = 0.208 R_{web} = 0.574

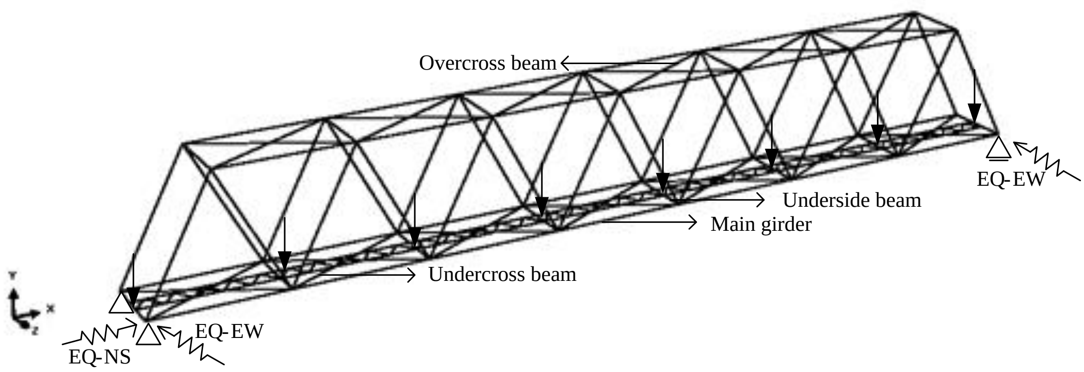


Figure 1. Full truss bridge model and loading condition

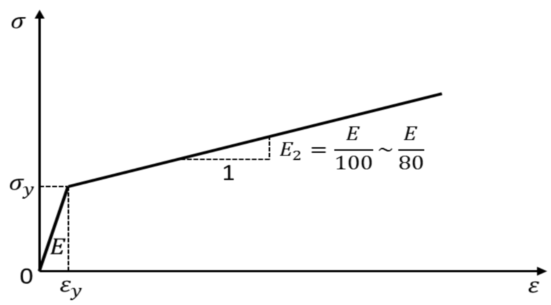


Figure 2. Bilinear stress-strain curve of SM490

Before carrying dynamic analysis, natural frequency of the structure has to be defined in order to get mass coefficient matrix and stiffness matrix, which were necessary to calculate damping matrix. Rayleigh damping was chosen. Natural frequency is defined by Eigen value analysis by displaying severe deformed modes of the structure. **Equations (1)**, **(2)** and **(3)** show the Rayleigh damping parameter formulations where α is the mass matrix constant, f_i and f_j are natural frequencies, h is damping constant, β is the stiffness matrix constant, C is Rayleigh damping matrix, M is mass matrix and κ is stiffness matrix. Damping constant is determined according to **Table 2** where constant for welded steel structure is 0.01.

$$\alpha = \frac{4\pi f_i f_j \left(hf_j - hf_i\right)}{f_j^2 - f_i^2} \qquad (1)$$

$$\beta = \frac{hf_j - hf_i}{\pi f_j^2 - \pi f_i^2} \quad (2)$$

$$C = \alpha M + \beta \kappa \quad (3)$$

Table 2. Typical Housner and current Japanese damping values

Type of structure	Percent damping
Piping	0.5
Welded steel	1.0
Structural steel building frames	2.0
Prestresses concrete	2.0
Reinforced concrete	5.0

Ground motion input data were taken from the recent severe earthquake in Japan, which were known as Hyogo prefecture (Kobe earthquake) in 1995, Tokachi earthquake (2004) and Tohoku earthquake (2011). Present study also employed El-Centro (1940) and Taft (1952) earthquake ground motions, which have been widely used as ground motion input data for design purposes. The acceleration respon spectra curves are shown through **Figure 3**. East-West (EW) earthquake was applied in the direction 3 transverse axis of bridge structure and North-South (NS) earthquake applied in the direction 1 longitudinal with the bridge axis for 2-dimension earthquake motion. For 1-dimension earthquake motion, only EW earthquake was applied in the direction 3 (**Figure 1**).

In the actual field, strength demand is always gained using dynamic response or static equivalent methods. Therefore, the present chapter analysis was performed in dynamic loading condition using the full truss bridge model.

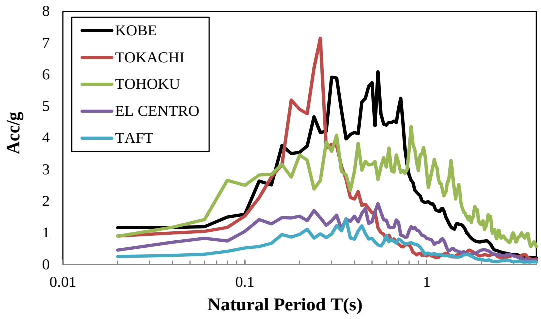


Figure 3. Acceleration response spectra

Most of bridge structures in the actual field always have a good capability in carrying the earthquake loads. Because they have been designed to carry the most severe earthquake. In here, the ground motion input data from some severe earthquakes (**Figure 3**) were magnified until the bridge structure exceeds its ultimate buckling point. In here, ultimate buckling stress and strain were compared with proposed ultimate stress and strain formulations in the previous paper by Susanti et. al., 2014..

3.RESULT AND DISCUSSION

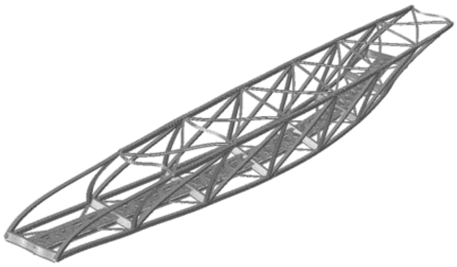
3.1Natural Frequency Analysis

Before carrying dynamic analysis, natural frequencies have to be calculated. According to the result analysis, it is found that biggest effective masses occurred in the mode 1 and mode 3 with 75.9% and 89.6% respectively. Hence, natural frequency values for dynamic analysis purposes are taken from those modes of structure. Deformed shapes of truss bridge structure from eigen analysis are shown in the **Figure 4**.

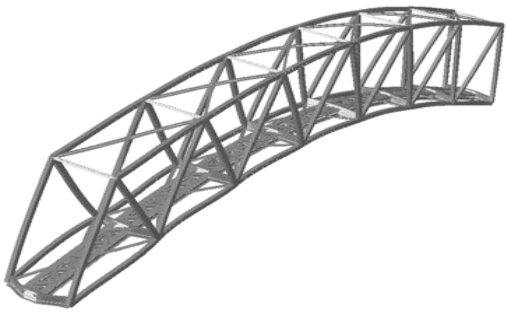
3.2 Response Analysis

3.2.1 1-Direction Earthquake Motion

This section is aimed to explore the influence of 1-direction earthquake ground motion to the stress and strain demand of truss bridge compression members



(a) First mode



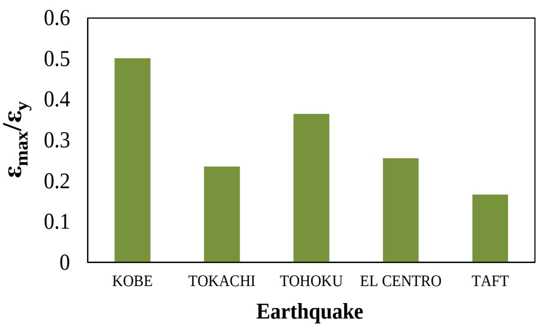
(b) Third mode

Figure 4. Vibration modes of truss bridge system

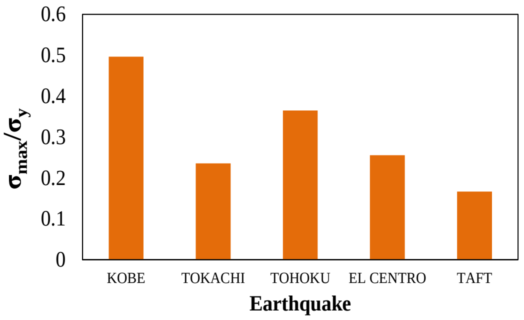
Figure 5 show the maximum stress and strain of truss bridge compression member for various ground motion data type including Kobe, Tokachi, Tohoku, El Centro and Taft earthquakes.

The maximum stress and strain demand were taken from the average compression stress and strain in the main girder members (**Figure 1**) which has most severe condition. In 1-direction earthquake motion, only EW earthquake that was applied in direction 3 transverse the bridge axis.

It was observed from the **Figure 5** that stress and strain demand for Kobe earthquake has highest magnitude followed by Tohoku, El Centro and Taft earthquake. Although, according to the earthquake acceleration response spectra in EW direction, Tokachi earthquake has bigger acceleration than Tohoku, but maximum stress and strain resulted from Tohoku earthquake has bigger magnitude. It may caused by longer earthquake period was occured in the Tohoku earthquake than Tokachi. Hence, beside the earthquake acceleration response spectra, earthquake period also has a significant effect to the maximum stress and strain result. Even El Centro earthquake that has much smaller acceleration results bigger maximum stress and strain than Tokachi earthquake due to its longger earthquake time period.



(a) Maximum stress

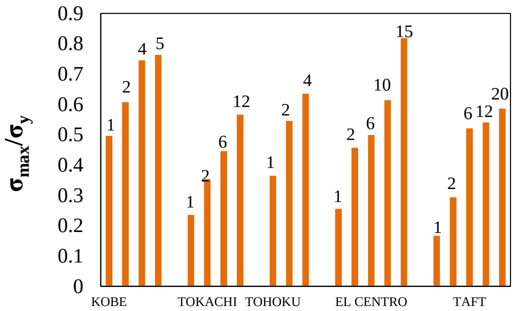


(b) Maximum strain

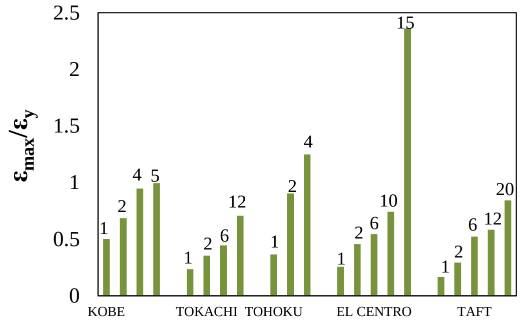
Figure 5. Maximum stress and strain of 1-dimension earthquake motion

When all of the earthquake ground motions data were magnified, different stress and strain escalation can be seen (**Figure 6**). All models with five earthquake motion show non-linear enchancement trend.

Number upside an each curve shows the earthquake magnification factor. Bigger magnification results bigger stress and strain. But the increasing of stress and strain is not linear following the earthquake magnification. After two times earthquake magnification, the increasing of stress and strain become more declivous. For compression members especially,it may caused by the structures have been yielding and exceed its buckling point.



(a) Maximum stress

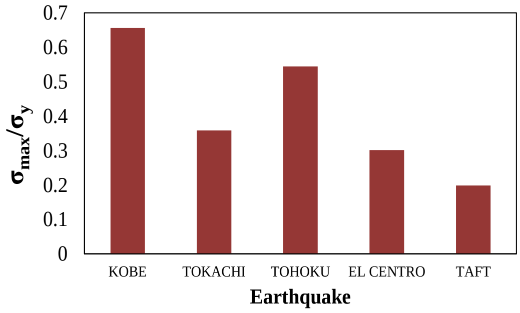


(b) Maximum strain

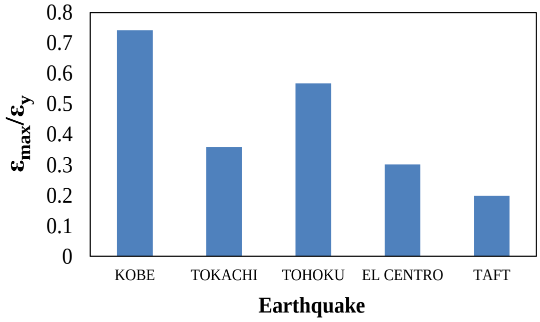
Figure 6. Earthquake magnitude vs. maximum stress and strain in 1-dimension earthquake motion

3.2.2 2-Direction Earthquake Motion

Second part of the present dynamic analysis is aimed to observe the influence of 2-dimension earthquake motion to the stress and strain demand. Since some references have proven that 2-dimension earthquake motion results more severe condition to the bridge system, this part investigated the significancy of 2-direction earthquake motion in decreasing the stress and strain demand compared with previous 1-dimension earthquake motion. Results of the analysis are presented in **Figure 7**. This figures show maximum stress and strain versus various earthquake ground motion. In here, Kobe earthquake resulted highest stress and strain demand, followed by Tohoku, Tokachi, El Centro and Taft. All earthquake data are in one time magnitude. None of them which has reached the yield regime.



(a) Maximum stress



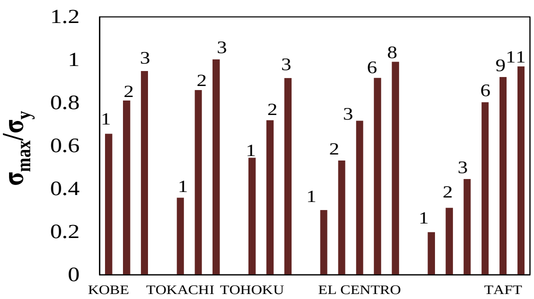
(b) Maximum strain

Figure 7. Maximum stress and strain of 2-direction earthquake motion

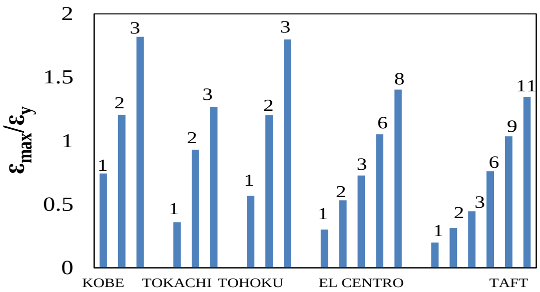
Figure 8 shows the escalation magnitude of five specified earthquake motion versus the average maximum stress and strain of the compression members. Similar as 1-dimension earthquake motion in the previous part, in here, the escalation trend of stres and strain was also in non-linear line. But in in 2-dimension earthquake motion, smaler magnitude than 1-dimension earthquake motion could produce bigger maximum stress and strain. The significant effect of 2-dimension earthquake motion to increase the maximum stress and strain compared to 1-dimension earthquake motion can be clearly seen in **Figure 9**. Especially for Kobe, Tokachi and Tohoku earthquakes, bigger gap can be found between 1-dimension and 2 dimension earthquake motion. Therefore, for big acceleration of earthquake motion, 2-dimension earthquake motion has more significant effect than in small acceleration of earthquake motion.

3.3 Result Verification

Since from the present results, it has been proven that 2-dimension earthquake motion has much more significant effect to increase the maximum stress and strain, therefore, present result verification used the results analysis from 2-dimension earthquake motion.

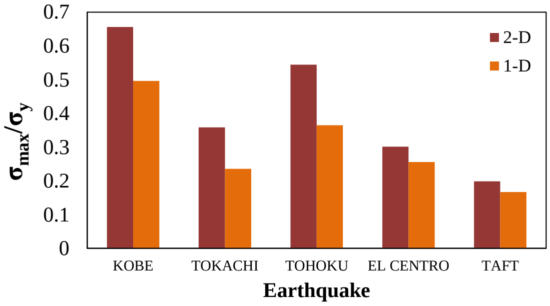


(a) Maximum stress

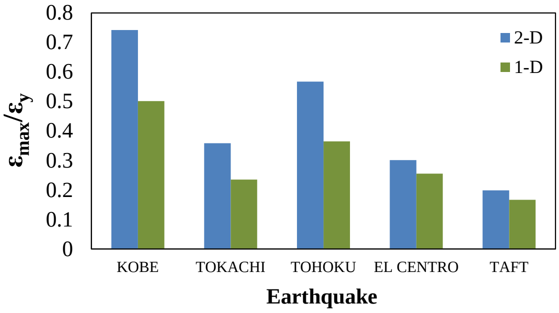


(b) Maximum strain

Figure 8. Earthquake magnitude vs. maximum stress and strain in 2-dimension earthquake motion



(a) Maximum stress



(b) Maximum strain

Figure 9. Comparison of 1-D and 2-D earthquake motion

Table 3. Average of maximum stress

Earthquake	σ_{max}/σ_y
Kobe	0.947
Tokachi	1.002
Tohoku	0.915
El Centro	0.991
Taft	0.970
	0.965

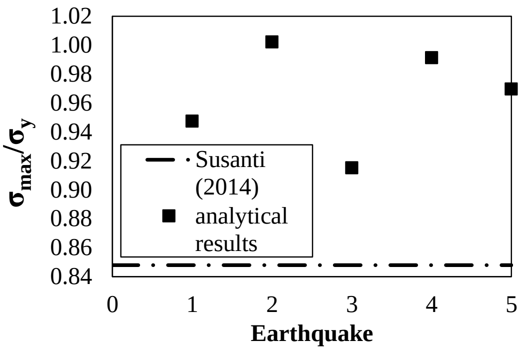


Figure 10. Maximum stress of present result compared with Susanti (2014) equation

Since the present main girders box section profile have width-thickness ratio as 0.366 and slenderness as 0.637, according to Susanti (2014) resulted normalized ultimate stress as 0.8479 for shell model.

Indonesian Standard (RSNI T-03-2005) proposed a formulae to determine the

Since the present main girders box section profile have a width-thickness ratio as 0.366 and a slenderness as 0.637, according to the Indonesian Standard normalized ultimate stress is 0.845. Therefore, it was regarded that present models have reached the ultimate buckling stress if the maximum stress has reached that value.

To determine the ultimate buckling stress and strain in the compression members, the effect of local buckling in the steel plate has to be considered. If the local buckling has been occured in the steel plate, the residual capacity of the structure become smaller and the plane hypothesis cannot be used. Since the present analysis only used beam element, the local buckling cannot be observed. That is why only global buckling that is considered during the analysis. The most severe buckling condition is occurred especially in the main girder located near the bearing supports.

Comparing the present results of buckling stress from dynamic analysis, it was proven that the equation taken from RSNI T-03-2005 has a conservative value. The average of maximum stress at the buckling point that was resulted from dynamic analysis is 0.965 (**Table 3** and **Figure 10**). It might be occured due to present dynamic analysis disregarded initial imperfection including initial displacement and residual stress. Therefore, the RSNI T-03-2005 equation has a safer value for design based purposes.

The result from Susanti (2014) is also has a conservative value compared with the present dynamic analysis results. Hence, the result of both RSNI-T-03 2005 and Susanti (2014) are suitable for design purposes.

utimate stress of a steel compression member (**Equation** (4)).

$$N_n\left(RSNI\right)=\begin{cases}\left[0.66^{\overline{\lambda}^2}\right]A_g f_y \rightarrow for\ \overline{\lambda}\leq 1.5\\ \dfrac{0.88}{\overline{\lambda}^2}A_g f_y \rightarrow for\ \overline{\lambda}>1.5\end{cases} \quad (4)$$

4. CONCLUSION

Present paper is aimed to investigate the influence of 1-direction and 2-direction earthquake motion to the maximum ultimate stress and strain of truss bridge compression members. Full railway truss bridge model in ABAQUS software was employed in the present dynamic analysis. According to present results, it can be conclude that:

i. Magnifying the earthquake acceleration can increase the stress and strain of compression member. The escalation of maximum stress and strain establish the non-linear line with the magnification factor.
ii. 2-Direction earthquake motion has much more significant effect in increasing the maximum stress and strain of the compression.
iii. members especially for high earthquake acceleration. Higher earthquake acceleration results more significant stress and strain escalation.
iv. The proposed ultimate stress of RSNI T-03-2015 and Susanti (2014) have conservative value compared with the present dynamic analysis results. Hence, it is suitable for design purposes.
v. Most severe buckling phenomenon occured in the members located near the bearing supports. Hence, the most dangerous part in the truss bridge structure is the members nearest with the bridges support.

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