HOKKAIDO UNIVERSITY

Title	A STUDY ON MODELING OF STRESS-STRAIN RELATIONSHIP AND PANEL POINTS OF STEEL TRUSS BRIDGES USED IN DYNAMIC RESPONSE ANALYSIS
Author(s)	MICHITANI, A.; ONO, K.; NISHIMURA, N.
Description	The Thirteenth East Asia-Pacific Conference on Structural Engineering and Construction (EASEC-13), September 11-13, 2013, Sapporo, Japan.
Relation	Proceedings of the Thirteenth East Asia-Pacific Conference on Structural Engineering and Construction (EASEC-13), September 11-13, 2013, Sapporo, Japan, G-3-6.
Citation	Proceedings of the Thirteenth East Asia-Pacific Conference on Structural Engineering and Construction (EASEC-13), G-3-6
Issue Date	2013-09-13
Doc URL	https://hdl.handle.net/2115/54424
Type	conference paper
File Information	easec13-G-3-6.pdf

Hokkaido University Collection of Scholarly and Academic Papers : HUSCAP

A STUDY ON MODELING OF STRESS-STRAIN RELATIONSHIP AND PANEL POINTS OF STEEL TRUSS BRIDGES USED IN DYNAMIC RESPONSE ANALYSIS

A.MICHITANI[1*], K. ONO[2†] and N. NISHIMURA[3]

[1] *Graduate School of Engineering, Osaka University, Japan*
[2] *Graduate School of Engineering, Osaka University, Japan*
[3] *Emeritus Professor, Osaka University, Japan*

ABSTRACT

It is required that the dynamic response analysis should be conducted as the method to evaluate the seismic performance of steel truss bridges according to Japanese seismic design specifications for highway bridges. On the other hand, it is known that the results of the dynamic response analysis are affected by the modeling of stress-strain relationship of steel and panel points of steel truss bridges. However, there are few studies on modeling the stress-strain relationship of steel and panel points used in the dynamic response analysis for evaluating the seismic performance of steel truss bridges.

In this study, modeling of stress-strain relationship and panel points of steel truss bridges is focused on and the dynamic response analysis is conducted against one of Level 2 earthquake ground motions. Based on the analysis results, the influence of modeling of the stress-strain relationship and panel points on the analysis results of steel truss bridges is investigated.

Keywords: stress-strain relationship, panel points, steel truss bridges, dynamic response analysis.

1. INTRODUCTION

The dynamic response analysis is required as the method to evaluate the seismic performance of the structure which behaves complexly when the earthquakes occur according to “Seismic Design Specifications for Highway Bridges” in Japan (Japan Road Association 2012). Steel truss bridge is one of the structures which behaves complexly in the earthquakes. On the other hand, it is known that the analysis results of the dynamic response analysis are affected by the modeling of stress-strain relationship of steel and panel points of steel truss bridges. However, there are few

* Presenter: Email: a_mich11@civil.eng.osaka-u.ac.jp

† Corresponding author: Email: k-ono@civil.eng.osaka-u.ac.jp

studies on the method how to set the stress-strain relationship of steel used in the dynamic response analysis for evaluating the seismic performance of steel truss bridges. It is reported that the axial force-axial displacement relationship of the steel members predominantly subjected to axial force like chords of steel truss bridges is asymmetry because of overall buckling before reaching yield stress in the compression stress side (Imamura et al. 2011). Therefore, it is necessary to reflect the influence of overall buckling on the stress-strain relationship of steel used in the analysis. Moreover, there are few studies on the influence of modeling of panel points on the analysis results of steel truss bridges though it is thought the stress resultant gained from the dynamic response analysis may be affected by modeling of panel points.

In this study, modeling of stress-strain relationship and panel points of steel truss bridges is focused on. In order to examine influence of modeling stress-strain relationship and panel points, the dynamic response analysis is conducted against one of Level 2 earthquake ground motions. Based on the analysis results, the influence of modeling of the stress-strain relationship and panel points on the analysis results of steel truss bridges is investigated.

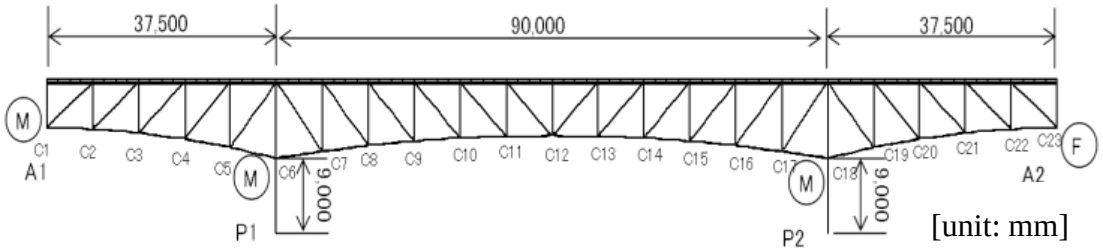


Figure 1: Upper-deck steel truss bridge

Table 1: Outline of truss bridge

Type of bridge	3 spans continuous upper-deck steel truss bridge
Bridge length	166.0m
Span length	37.5m＋90.0m＋37.5m
Width	9.75m

Table 2: Boundary condition of bearing

	Longitudinal direction	Transverse direction	Vertical direction
A1,P1,P2	Free	Fix	Fix
A2	Fix	Fix	Fix

2. ANALYSIS MODEL AND ANALYSIS CONDITION

2.1. Analysis Model

A target bridge of this study is an Upper-deck steel truss bridge (Japanese Society of Steel Construction 2011). Outline of target truss bridge is shown in Table 1. The general view of the bridge is shown in Figure 1. The whole bridge was modeled using the fiber model (Earthquake Engineering Research Center Inc. 2007) for all steel members of the steel truss bridge. “P1” and “P2” in Figure 1 are RC bridge piers. These RC piers were modeled with the elastic beam element.

As a principle, the analysis model was set with reference to models shown in the previous research (Japanese Society of Steel Construction. 2011). However thickness of steel plates of webs and flanges in lower chord members, upper chord members, vertical members and diagonal members were increased compared with the model in the previous study in order to prevent local buckling of steel panels. Therefore, the values of width-thickness ratio parameter R_R of lower chord members, upper chord members and vertical members are less than 0.7. The values of slenderness ratio parameter $\bar{\lambda}$ of them exist from 0.15 to 0.4. The definitions of parameters mentioned above are given as equations (1) and (2).

$$\bar{\lambda} = \frac{1}{\pi}\sqrt{\frac{\sigma_y}{E}}\frac{l}{r} \tag{1}$$

$$R_R = \frac{b}{t}\sqrt{\frac{\sigma_y}{E}\frac{12\left(1-\nu^2\right)}{4\pi^2}} \tag{2}$$

where l = effective buckling length of column; r = radius of gyration of cross section; σ_y = yield stress; E = Young's modulus; ν = Poisson's ratio; b = flange width; t = plate thickness;

Boundary conditions of bearings of the truss bridge are shown in Table 2. An example of cross section of main members is shown in Figure 3. Web plates and flange plates of the cross sections were divided into ten segments as shown in Figure 3 respectively. Main members were divided into four elements in the axial direction. The division of cross sections and element has to be investigated in detail in the feature works.

2.2. Panel Points

It is thought that the rigidity of panel points exists between the rigidity of pin connection and that of rigid connection. However, it is difficult to set the rigidity of panel points in an appropriate manner. Therefore, two types of panel points were applied to points between chord members and web members. Two types are a rigid connection shown in Figure 4(a) and a pin connection in Figure 4(b).

2.3. Stress-Strain Relationship

Two types of stress-strain relationship of steel were applied to upper and lower chord members, diagonal members and vertical members.

One of them is an asymmetry bilinear model shown in Figure 5(a). The other is a linear model shown in Figure 5(b). The asymmetry bilinear model was set in consideration of the nonlinearity by overall buckling. In Figure 5, σ_y and σ_u correspond to yield stress and buckling strength of column respectively. ε_{at} and ε_{ac} are allowable values of strain for evaluating seismic performance. ε_{at} and ε_{ac} have to be investigated in detail in the feature works. σ_u was calculated according to the strength curve for welded steel columns with box section specified in the 2012 design specifications (Japan Road Association 2012). σ_u is calculated using the following equation (3). Moreover, N_u and M_u

results are calculated using following equations (4) and (5) respectively. *A* and *W* correspond to area of cross section and section modulus.

$$\frac{\sigma_u}{\sigma_y} = \begin{cases} 1.0 & (\bar{\lambda} \le 0.2) \\ 1.059 - 0.258\bar{\lambda} - 0.19\bar{\lambda}^2 & (0.2 < \bar{\lambda} \le 1.0) \\ 1.427 - 1.039\bar{\lambda} + 0.223\bar{\lambda}^2 & (1.0 < \bar{\lambda}) \end{cases} \quad (3)$$

$$N_u = \sigma_u \times A \quad (4)$$

$$M_u = \sigma_u \times W \quad (5)$$

2.4. Dynamic Response Analysis Condition

The design earthquake ground motion (II-I-1) prescribed in the seismic design specifications (Japan Road Association 2012) in Figure 6 was applied in dynamic response analysis. The ground motion was input in both the longitudinal direction and transverse direction.

In the analysis, the self-weight of the structure was applied at first, and then dynamic response analysis was carried out in consideration of the geometrical nonlinearity.

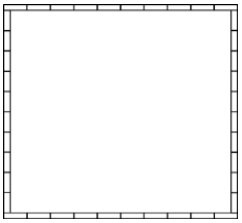
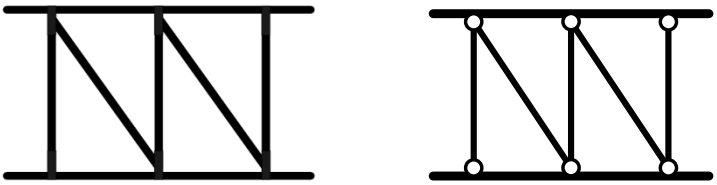
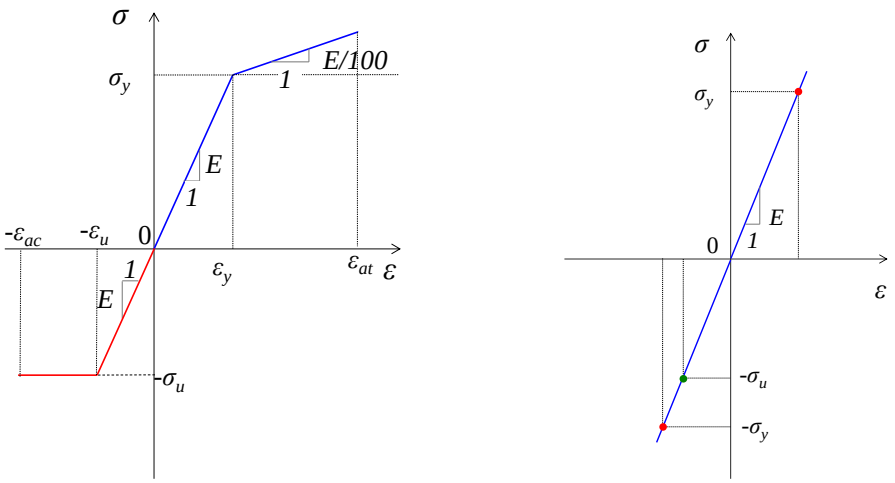


Figure 3: Cross section of main members



(a) Rigid connection **(b) Pin connection**

Figure 4: Model of panel points



(a) Asymmetry bilinear model **(b) Linear model**

Figure 5: Modeling of the stress-strain relationship

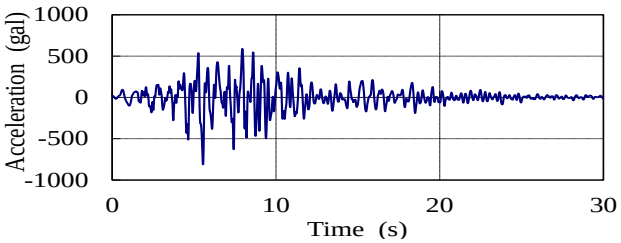


Figure 6: Design earthquake ground motion (II-I-1)

Table 3: Results of eigenvalue analysis

	Rigid connection		Pin connection	
Natural vibration mode number	1	7	1	7
Natural period(s)	0.662	0.202	0.662	0.205

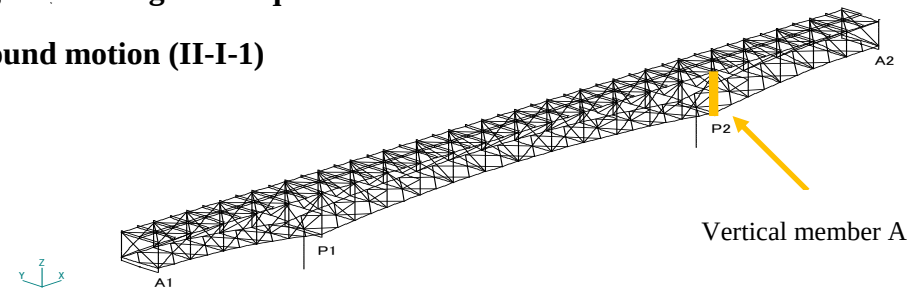


Figure 7: Vertical member A

In the dynamic response analysis, Reyleigh damping was applied and the value of the damping ratio is 0.01 for all steel members. The results of eigenvalue analysis are shown in Table 3. Based on the results of eigenvalue analysis, two modes shown in Table 3 were adopted for setting Reyleigh damping.

3. ANALYSIS RESULTS

In many cases of analysis results, analysis results for transverse direction are larger than those for longitudinal direction. Therefore, analysis results for transverse direction are shown below.

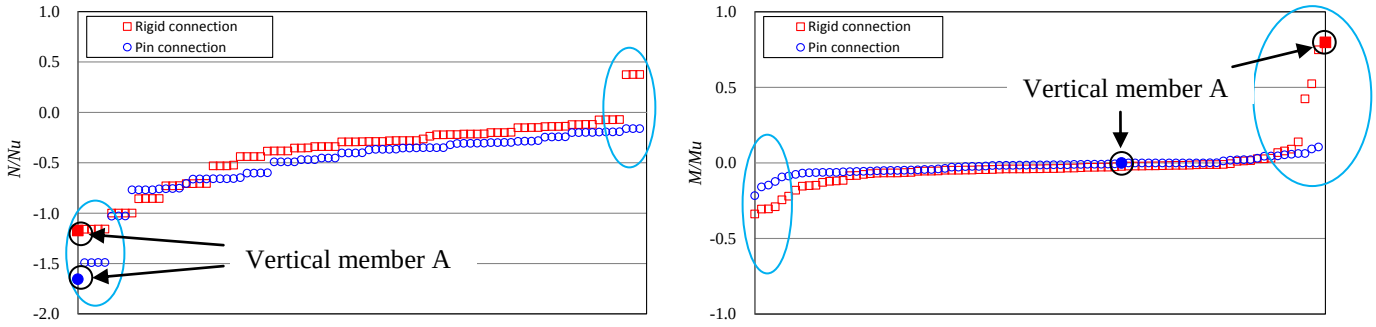
Difference of panel points is focused on. Maximum absolute values in respective axial force N and bending moment M for all vertical members were picked out. Then N/N_u and M/M_u are arranged in increasing order of values respectively. Examination of stress resultant was conducted mainly by absolute values. Distribution of stress resultant in linear model is shown in Figure 8. That in asymmetry bilinear model is shown in Figure 9. In Figures 8 and 9, there is obvious difference between rigid connection and pin connection in the stress resultant enclosed with a circle. Regarding linear model, it is found that the compressive axial force of pin connection is larger than that of rigid connection as shown in Figure 8(a). Figure 8(b) indicates that M/M_u of rigid connection is larger than that of pin connection. Regarding asymmetry bilinear model, it is found that distribution of N/N_u is little influenced by modeling of panel points as shown in Figure 9(a). However, Figure 9(b) shows that M/M_u of rigid connection is larger than that of pin connection.

Difference of model of stress-strain relationship is focused on. Distribution of stress resultant of pin connection is shown in Figure 10. That of rigid connection is shown in Figure 11. In Figures 10 and 11, the stress resultant focused on is enclosed with a circle like Figure 8. It is found that the

compressive axial force of linear model is larger than that of asymmetry bilinear model as shown in Figure 10(a) and Figure 11(a). Regarding pin connection, Figure 10(b) shows that distribution of M/M_u is little influenced by modeling of stress-strain relationship. However, Figure 11(b) indicates that M/M_u of asymmetry bilinear model is larger than that of linear model.

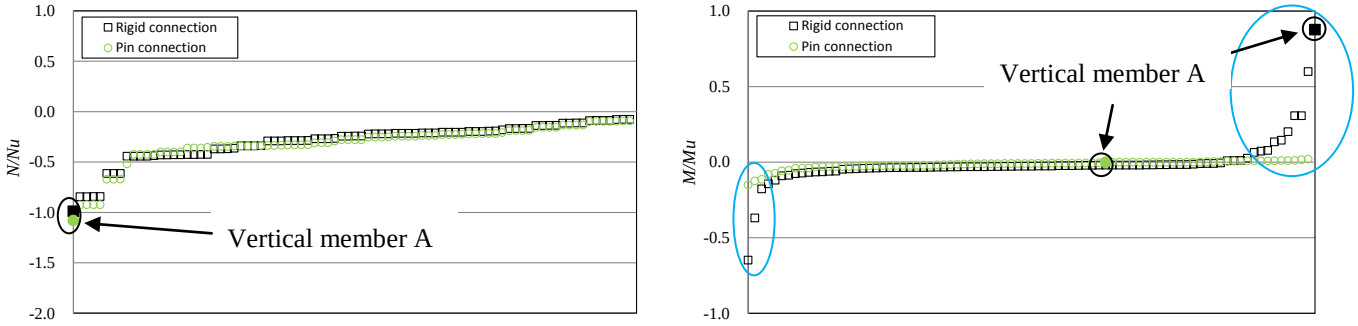
Analysis results of 'Vertical member A' shown in Figures 8~11 are focus on. In Figures 8~11, stress resultant of 'Vertical member A' is expressed by solid marks (■, ■, ●, ●). 'Vertical member A' is shown in Figure 7. Response history of N/N_u in linear model is shown in Figure 12. Response history of M/M_u in asymmetry bilinear model is shown in Figure 13. As shown in Figure 12(a), it is found that axial force of pin connection is larger than that of rigid connection like Figure 8(a). On the other hands, regarding asymmetry bilinear model, Figure 13(a) indicates that N/N_u is little influenced by modeling of panel points like Figure 9(a). Figure 12(b) and Figure 13(b) show that M/M_u is hardly generated as for pin connection although M/M_u is generated regarding rigid connection like Figure 8(b).

The response history of σ/σ_y-$\varepsilon/\varepsilon_y$ in 'Vertical member A' is shown Figure 14. In linear model, Figure 14(a) shows that both stress of pin model and that of rigid model reach σ_u. Moreover, it is found that σ/σ_y of rigid model is large than that of pin model. This is caused by influence of M as shown in Figure 12(b). On the other hands, in asymmetry bilinear model, Figure 14(b) indicates that rigid model reach σ_u and that plastic strain appears. This is caused by influence of M as shown in Figure 13(b).



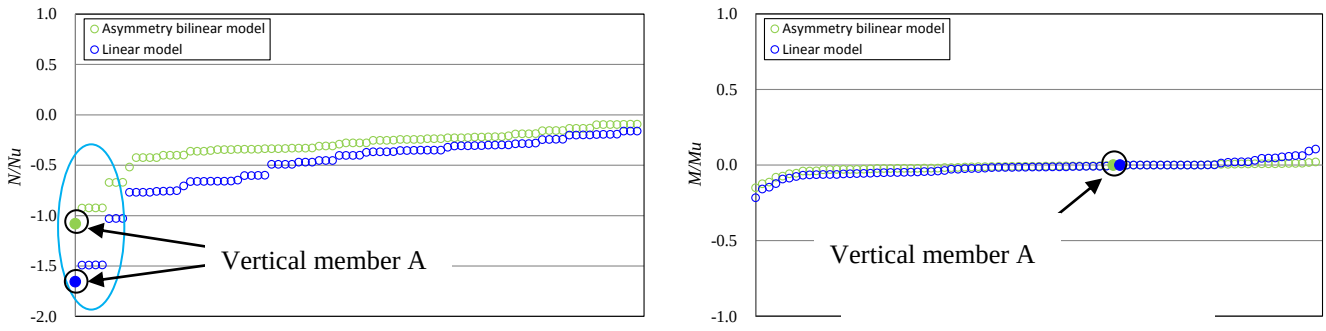
(a) Distribution of N/N_u **(b) Distribution of M/M_u**

Figure 8: Distribution of stress resultant (Linear model)



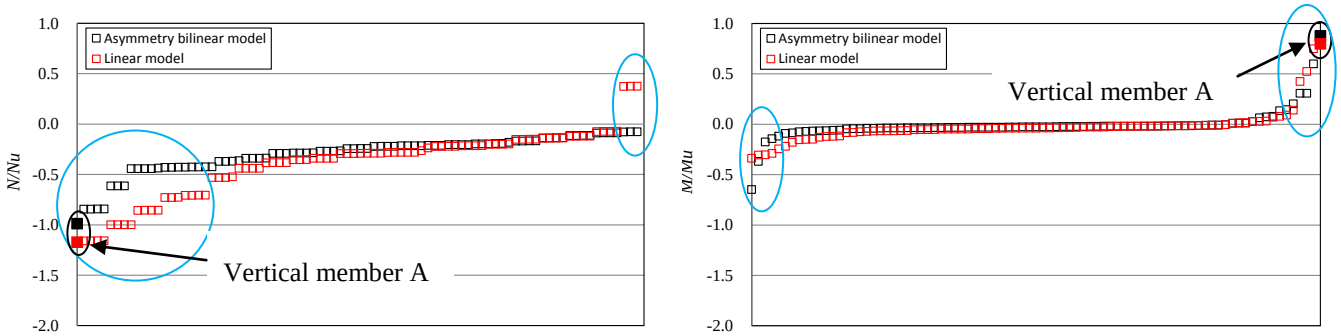
(a) Distribution of N/N_u **(b) Distribution of M/M_u**

Figure 9: Distribution of stress resultant (Asymmetry bilinear model)



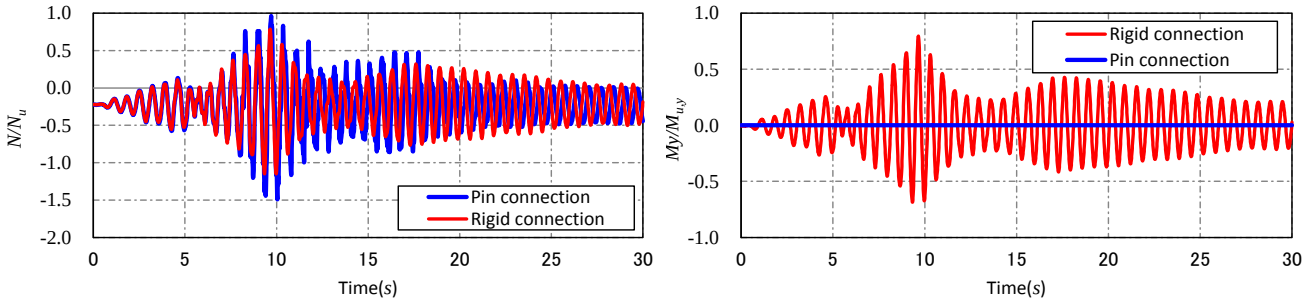
(a) Distribution of N/N_u **(b) Distribution of M/M_u**

Figure 10: Distribution of stress resultant (Pin connection)



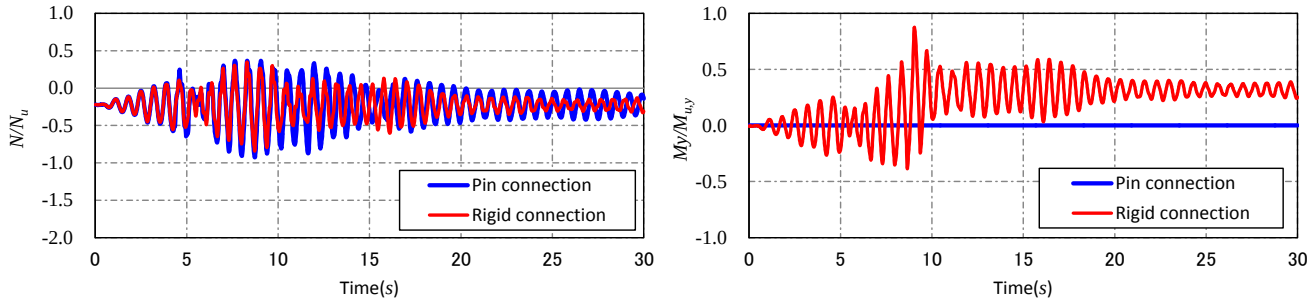
(a) Distribution of N/N_u **(b) Distribution of M/M_u**

Figure 11: Distribution of stress resultant (Rigid connection)



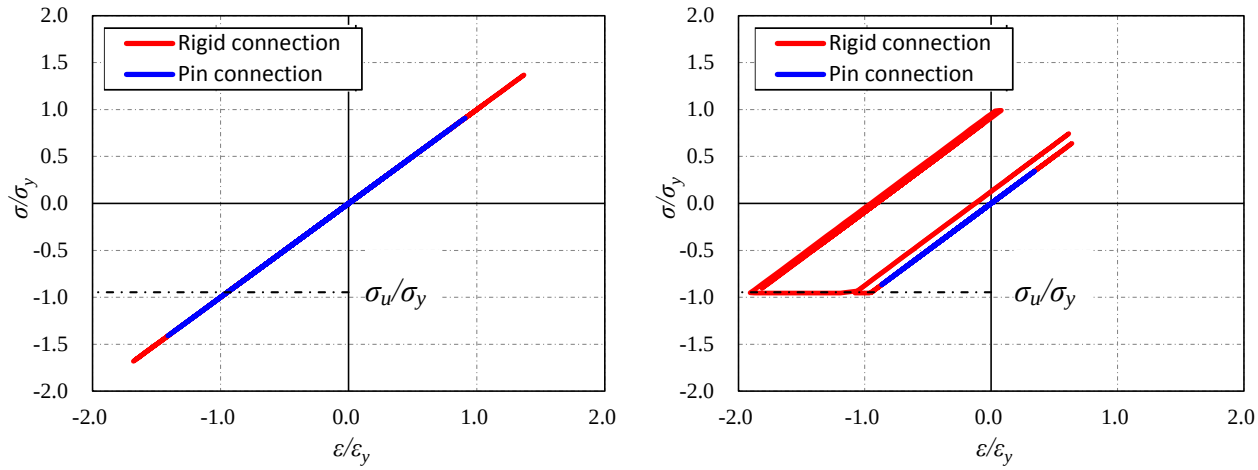
(a) Response history of N/N_u **(b) Response history of M/M_u**

Figure 12: Response history of vertical member A (Linear model)



(a) Response history of N/N_u **(b) Response history of M/M_u**

Figure 13: Response history of vertical member A (Asymmetry bilinear model)



(a) Linear model **(b) Asymmetry bilinear model**

Figure 14: Response history of σ/σ_u-$\varepsilon/\varepsilon_u$

4. CONCLUSIONS

In this study, modeling stress-strain relationship and panel points of steel truss bridges is focused on. In order to examine influence of modeling stress-strain relationship and panel points, the dynamic response analysis is conducted against one of Level 2 earthquake ground motions. Based on the analysis results, it is found that axial force is hardly influenced by modeling of panel points as for asymmetry bilinear model although axial force and bending moment are influenced by modeling stress-strain relationship and panel points of steel members except axial force by asymmetry bilinear model.

5. ACNOWLEGEMENTS

This work was supported by JSPS KAKENHI Grant Number 25420483. The authors acknowledge Dr. Nonaka T. and Mr. Mogoshi K. (Earthquake Engineering Research Center Inc.) for their valuable advises.

REFERENCES

Imamura T., Ono K., Taniue H., Matsumura M., Yoshiyama J. and Nishimura N. (2011) Study on load carrying capacity and ductility of steel members predominantly subjected to axial force by elasto-plastic finite displacement analysis, Japan Society of Civil Engineers Annual Meeting, Vol.66, I-641, pp.1281-1282.

Earthquake Engineering Research Center Inc. (2007). EERC/Fiber User' s manual .

Japan Road Association (2012). Specifications for highway bridges, Part II; Steel Bridges.

Japan Road Association (2012). Specifications for highway bridges, Part V; Seismic Design.

Japanese Society of Steel Construction (2011). Current status and improving the reliability of dynamic seismic analysis of steel bridges with fiber model.