Evaluation of Safety Margin on Steel Through-truss Bridges in After-fracture State

Masamichi SAITO

Steel & Hybrid Structures Laboratory, Structures Technology Division

Manabu IKEDA

Steel & Hybrid Structures Laboratory, Structures Technology Division (Former)

Steel truss bridges consists of many different trussed members and it is important to clarify the possibility of bridge collapse following the fracture of a member. This study evaluates the safety margin on a standard steel railway through-truss bridge after the fracture of certain members, using 3-D finite element analysis. Results showed that the safety margin remains fairly high with surviving members after the fracture of lower chord members, because floor members can make up for the lost sectional force from the fractured members. Finally, the degree of importance of each member was evaluated.

Keywords: *through truss, member fracture, redundancy, fracture critical member, finite element analysis*

1. Introduction

Over the past few years, there have been several reports of steel members fracturing on road bridges, including cases where this has led to bridge collapse. For example, a steel truss bridge collapsed due to the fracture of a member on a panel point on its main truss [1]. In another case, a diagonal member on a steel truss bridge fractured but did not lead to the bridge's collapse [2]. In reaction to these reports, the influence of member fracture on bridge collapse was examined mainly for road steel-truss bridges [3]. "Redundancy" is defined as the load bearing capability of a bridge after the fracture of its members, namely the safety margin that exists before bridge collapse [4].

In terms of steel railway bridges in Japan, there have been no reported bridge collapses due to member fracture. However, fractured members or other similar damage have been found. Although it is a prerequisite during normal structural maintenance work to take measures at an early stage, which includes checking for member damage, it is necessary to minimize risks of bridge collapse or influence on the running safety of trains even if a member become fractured. It is therefore important to be able to know the remaining safety margin of a bridge in terms of its collapse, and the degree to which running safety has been affected after a member has become fractured, in order to ensure a higher level of redundancy of structures.

Once the influence of a bridge member fracture on bridge collapse and running safety have been clarified, members can be ranked in order of importance and during maintenance, possible countermeasures to be applied to each bridge member can be evaluated. The safety factor in design for each member at present is the same regardless of where it is located in a structure. If the safety factor is adjusted according to the identified importance of the member, i.e. a higher or lower safety factor set for members whose fracture would lead to a higher or lower possibility of bridge collapse, designs would be more efficient and bridge safety against collapse could be improved, inter alia.

Three-dimensional finite element analyses of a bridge with a standard span and assuming fractured members was performed for this study, in order to understand the level of redundancy on an existing steel railway through-truss bridge with a fractured member [5]. The remaining safety margin before bridge collapse was evaluated using verification values in the members surrounding the fractured member. Finally, the members identified as possibly leading to bridge collapse if they fractured, were extracted. In addition, the influence of the difference in the train load and the state of corrosion of members on the resultant safety margin was also investigated.

2. Assumed member fracture and influence on safety

Table 1 shows the target members and associated assumed fracture events for three typical steel railway bridge structures and the influence on bridge collapse and running safety of each fractured member. When a member fractures on a steel railway bridge, it is important not only to ensure safety from bridge collapse but also to secure running safety. The assumed fractures in the table were not only reported fractures but also those resulting from the fracture of a member.

The category "large influence" in Table 1 indicates a high possibility of bridge collapse resulting from the fractured member, which is particularly true for the main girder of deck girders and through girders. Redundancy in these cases is deemed to be low. The floor system of through girders, etc. however was considered to have a "small influence" on the bridge collapsing. This indicates that the possibility of the girder itself collapsing is low even if the floor system is broken.

The main truss on through truss bridges is composed of multiple members and the possibility of the bridge collapsing from a fracture in one of its members is assumed to be relatively low. Nevertheless, the influence of a fractured member on the safety margin is not clear. To clarify the

Table 1 Influence of member fracture on bridge collapse and loss of running safety

Form of structure	Location of fracture	Assumable type of fracture	Influence on bridge collapse	Influence on loss of running safety
Deck girder	Main girder	<ul style="list-style-type: none"> • Severe corrosion • Propagation and penetration of fatigue crack • Buckling (local, global) • Fracture of connection 	Large influence	Large influence
Through-girder	Main girder		Large influence	Large influence
	Floor system (Stringer, Floor beam)		Small influence	Unknown
Through-truss	Main truss		Unknown	Unknown
	Floor system (Stringer, Floor beam)		Small influence	Unknown

remaining safety margin against bridge collapse, which is the most serious event for a bridge, this paper focuses on the influence of a fracture in the member of a through truss bridge on its failure. The impact of a fracture in a member of the floor system on train safety, will be reported separately.

3. Analysis of bridge behavior in after-fracture state

3.1 Target structure

The target bridge of this study was a single-track simple through-truss bridge with a span of 62 m and an open-type floor system composed of stringers and floor beams. The outline of the target bridge is shown in Fig. 1. This bridge was based on a standard design used in the former Japan National Railways (WTT862). Its design load is “KS-18” which means a maximum axle load of 180 kN (18 tons). The members were verified with the allowable stress design method. The main truss members, such as upper chords, lower chords and diagonal members, were designed as axial force members and thus the bending moment was not taken into consideration in the design stage. In addition, the panel points of the truss were composed of two gusset plates.

Fig. 1 Outline of target bridge and location of fracture

3.2 Procedure of evaluation

The evaluation of redundancy in this study was carried out in the procedure shown in Fig. 2.

First, scenarios such as the location and failure mode of the assumed member fracture were set. Next, finite element analyses were carried out assuming presence of and absence of a member fracture, and then the sectional force of the members was calculated. The safety margin following a fracture in the member was verified by checking whether the sectional force of the members surrounding

Fig. 2 Evaluation procedure

the fractured member was less than the sectional capacity. When the safety margin in the surrounding members following a fracture was small, it was deemed, in this study, that there was a high possibility that a chain of fractures would result, leading to the collapse of the bridge. Therefore, if the surrounding members fractured along with the target member, that target member was deemed to be a fracture critical member (FCM) [4].

3.3 Member fracture scenarios

In this study, the following three cases were set assuming the fracture of each member of the main truss.

(1) Fracture of tensile members

Fractures of lower chord members and tensile diagonal members, are assumed to occur under a tensile load when a train is passing through, when for example its cross section decreases due to corrosion.

(2) Fracture of compressive members

Fractures of upper chord members and compressive diagonal members, are assumed to occur due to the buckling of the member under compressive forces when a train is passing through, when for example its cross section decreases due to corrosion.

(3) Fracture of panel points

Fractures in the gusset plates on panel points, are assumed to occur due to failures in the rivets under force exerted from main truss members when a train is passing through, when for example its cross section decreases due to corrosion.

There is also the possibility of the sectional force exceeding that obtained in the static analyses due to dynamic effects because the sectional force acting on the member before the fracture is abruptly released on rupture [4]. In

this study, as described below, this increased sectional force was considered to be an impact force, which was obtained by multiplying the impact factor and the static sectional force. This impact force was taken into consideration in cases where the fracture occurred on the tensile members. For the compressive members where it was assumed that fractures could occur through buckling, this impact force was not taken into consideration.

3.4 Outline of the analysis

The target bridge was modeled using a three-dimensional finite element method. The analysis model is shown in Fig. 3. All main truss members were modeled with linear elastic beam elements. Although the connection of truss members at panel points were assumed to be pinned connections as in a normal bridge design, the connections in the model were assumed to be fixed connections in consideration of the structure of the panel points on the target bridge, etc. Furthermore, in order to confirm the influence of the modeling method on the analysis results, the open type floor system, consisting of stringers and floor beams, was modeled in two different ways: beam elements in one model and shell elements in another model.

For the bearings, fixed and movable conditions were set in consideration of the neutral axis height of the lower chord member and the bearing height. Track members were not represented in the analysis model. The analysis carried out was linear static analysis. A general-purpose finite element analysis program MSC Nastran 2012.1 was used for this analysis.

The conditions of the experiment are shown in Table 2. Three cases were examined: a dead load exerting force

without damaged members; a train load exerting force without damaged members; and a case where the impact force from the fractured member was considered, as shown in Fig. 4. The analyses were conducted to represent the case of a fracture on each member of the main truss, as shown in Fig. 1.

Table 3 shows the resulting evaluation of each case. The train load was set at KS-18 (maximum axle load 180 kN) as the design load in Case 1. In Case 2, the train load was set at KS-14 (maximum axle load 140 kN) corresponding to the load of a real train, to confirm the influence of the tolerance in the design load. Case 3 was a variation of Case 2, with the same train load but with a different level of corrosion on the panel point. The thickness of the gusset plates was assumed to be uniformly half their original thickness.

The train load was set to maximize the axial force on the target member in each case. The value of the train load was increased to 1.30 to 1.33 times its original value depending on the location of the target member, in consideration of the impact load from the running train.

The sectional force F_{di} at member i after the fracture of member k was calculated by (1), i.e., the sum of the

Fig. 4 Loading method for each step

Table 3 Case of redundancy evaluation

Case	Train load	Corrosion of panel points
Case 1	KS-18	No corrosion
Case 2	KS-14	No corrosion
Case 3	KS-14	With corrosion

Fig. 3 Three-dimensional finite element model of target bridge

Table 2 Conditions of action

Analysis step	Action	Status of members	Method of loading
Step 1	Dead load	No damage	Gravity acting on mass of members
Step 2	Train load	No damage	Axle position for maximizing axial force on target member
Step 3	Impact load from member fracture	Target member* is separated.	Sectional force is applied in reverse direction with an impact factor i

* Target members are shown in Figs. 1 and 3(b).

sectional force obtained in each analyzed case is shown in Table 2.

$$F_{di} = F_{Di} + F_{Li} + i \cdot F_{fi} \quad (1)$$

where,

F_{Di} : sectional force of member i under a dead load with no damage (Step 1)

F_{Li} : sectional force of member i under a train load with no damage (Step 2)

i : the impact factor of the sectional force of member i after the fracture of member k

F_{fi} : sectional force of member i in the model with separated member k (Step 3), to which the sectional force on member k with no damage (Step 1 and 2) is applied in the reverse direction.

Strictly speaking, the impact factor i largely depends on the length of time between fracture occurrence to release of the sectional force, and yielding of the surrounding members, etc. However, it is difficult to determine the impact factor unconditionally. In this study, with consideration for a good safety margin [6], the impact factor was set uniformly on the basis of reference [7]. In reference [7], the impact factor was calculated using the dynamic response ratio obtained from the dynamic analyses of a single-degree-of-freedom (SDOF) model. In the SDOF model, the dynamic response displacement $u(t)$ at time t when the step load P_0 instantaneously acts, is given by (2).

$$\frac{u(t)}{(u_{st})_0} = 1 - e^{\zeta \omega_n t} \left(\cos \omega_n t + \frac{\zeta}{\sqrt{1 - \zeta^2}} \sin \omega_n t \right) \quad (2)$$

Where,

$(u_{st})_0$: static displacement ($=P_0 / k$)

k : spring constant of the SDOF model

ζ : damping constant

When the damping constant $\zeta = 0.05$, maximum dynamic response magnification is 1.854. Therefore, in this study, it was decided to use this number as the impact factor i . It should be noted that the impact factor i was set to 1.0, in the case of a fracture from compressive force.

4. Evaluation results for case with no damage

Results for the cases where there was no damage (dead load + train load) are shown below. An evaluation of member i and its panel points on the both ends was carried out using (3), in the same manner as with the design standard.

$$\gamma_i \frac{S_{di}}{R_{di}} \leq 1.0 \quad (3)$$

where,

S_{di} : sectional force of member i or its panel points on the both ends under a dead load and a train load with no damage

R_{di} : sectional capacity of member i or its panel points on both ends

γ_i : the structure factor, in this study: 1.0

Axial force, in-plane bending, out-of-plane bending and their coupling were considered for the verification of each member. In tensile members, axial plastic capacity was set as the axial capacity. In compressive members, the axial

capacity was set to which ever value was lower between axial plastic capacity and buckling capacity.

For the panel points, the capacity was calculated in the fracture modes 1 to 7, listed below. The calculation method of each capacity was based on reference [4] therefore details of this method are not given here. In the design standard, panel points are not directly verified using the sectional force because the thickness is set above the minimum thickness determined by the axial force of diagonal members. However, in this study, the panel points were verified directly in detail because there was a possibility that the capacity of the panel points was smaller than that of the truss members due to the decrease in thickness from corrosion.

Mode 1: shear fracture of the rivet

Mode 2: yielding or fracture of the outer edge of the gusset plate

Mode 3: block shear fracture of the gusset plate

Mode 4: yielding or fracture of the diagonal members, etc.

Mode 5: local buckling of the plate at the tip of the compressive member

Mode 6: shear yield of the gusset plate

Mode 7: buckling of the area around the gusset plate surrounded by its free edges, diagonal members and chord members etc.

Figure 5 shows the ratio of the capacity of the panel points calculated in Modes 1 to 6 to the capacity of the diagonal member. Mode 7 capacity is omitted in this study because this item its purpose was only to serve as verification of the width-thickness ratio of the gusset plate and the evaluation showed a good safety margin with respect to buckling. From Fig. 5, it can be seen that Mode 1 displays minimum capacity if the gusset is not corroded (Cases 1 and 2 in Table 3). Moreover, the values were below 1.0 in the majority of panel points which means that the capacity of the panel points was smaller than that of the diagonal members. This means in turn that if an external force exceeding the design limit is applied, the panel points may

Fig. 5 Capacity of panel points (no loss of thickness)

fracture ahead of the diagonal members. The reason for this is thought to be due to the difference in the manner of determining the dimension of the cross section at the design stage. The size of the cross section of the diagonal members is set based on the Allowable Stress Method whereas the cross-section for the panel point is determined in consideration of the minimum thickness, as mentioned above.

The verification result (left hand side of eq. (3)) in the case where there was no damage was approximately 0.5 ~ 0.6. This is because the members were designed applying the Allowable Stress Method and the verification this time was based on the fracture capacity as the limit value while adopting a safety factor value of 1.0 for verification.

5. Results of safety margin evaluations in after-fracture state

5.1 Difference in the floor system modeling method

Figure 6 compares the verification values (left-hand side of eq. (3)) for two different floor system models. This verification value indicates the maximum verification values for surrounding members at the time of each member fracture. Figure 6 shows that modeling with beam elements gives design-specific values, based on which an evaluation can be made of the sectional force in members with a safety margin equal to or slightly above those obtained through modeling with shell elements. These results confirmed that the open type floor system can be modeled using beam elements without any problem.

Fig. 6 Comparison of floor system modeling method

5.2 Safety margin in after-fracture state

As an example of results from the analyses, Fig. 7(a) shows the case where the lower chord 2 fractures, and Fig. 7(b) shows a fracture of diagonal member 2. The numbers of the members are shown in Fig. 1. Both figures show the

deformation diagram showing the moment of fracture and the verification values (left-hand side of eq. (3)) for the surrounding members. The verification values are color coded for each range corresponding to each 0.5 increment.

If the verification value exceeds 1.0 in some surrounding members, it is assumed that the possibility of knock-on fractures is high, which would lead to the bridge collapsing.

After the lower chord 2 fractured, the verification values of each member were lower than 1.0. It is considered that this was because the axial force borne by the lower chord 2 before the fracture was transmitted to the other part of the truss, such as the lower chords of the main truss on the opposite side from the fractured member, via the lower lateral braces, stringers and floor beams. On the other hand, when the diagonal members fractured, the verification values grew and exceeded 2.0 in the lower and upper chords. The verification values exceeded 1.0 in the diagonal members, the upper chords and diagonal members of the main truss on the opposite side.

Maximum verification values in the surrounding members

Fig. 7 Examples of verification results

Fig. 8 Verification results

Table 4 Maximum verification value of surrounding members and their location

Result of verification Fractured member		Case 1		Case 2		Case 3	
		Maximum verification value	Location of maximum verification value	Maximum verification value	Location of maximum verification value	Maximum verification value	Location of maximum verification value
Upper Chord	1-1	1.33	Diagonal member 3-2	1.22	Diagonal member 3-2	1.22	Diagonal member 3-2
	2-1	1.02	Upper chord 3-2	0.91	Upper chord 3	1.06	Panel point L2
	3-1	1.10	Upper chord 3-2	0.98	Upper chord 3	0.98	Upper chord 3-2
Lower Chord	1-1	0.58	Upper chord 1-2	0.47	Upper chord 1-2	0.47	Upper chord 1-2
	2-1	0.61	Upper chord 1-2	0.53	Panel point L2	1.06	Panel point L2
	3-1	0.64	Upper chord 3-2	0.55	Lower chord ① -1	1.06	Panel point L2
	4-1	0.69	Upper chord 3-2	0.58	Upper chord 3-2	1.05	Panel point L2
Diagonal Member	1-1	1.52	Diagonal member 3-2	1.41	Diagonal member 3-2	1.41	Diagonal member 3-2
	2-1	3.26	Diagonal member 3-2	3.05	Diagonal member 3-2	3.05	Diagonal member 3-2
	3-1	1.37	Diagonal member 2-1	1.29	Diagonal member 2-1	1.29	Diagonal member 2-1
	4-1	1.98	Lower chord 2-1	1.94	Lower chord 2-1	1.94	Lower chord 2-1
	5-1	1.41	Diagonal member 4-1	1.33	Diagonal member 4-1	1.33	Diagonal member 4-1
	6-1	0.87	Diagonal member 5-2	0.75	Diagonal member 5-2	0.75	Diagonal member 5-2
	7-1	0.54	Diagonal member 5-2	0.47	Diagonal member 5-2	0.47	Diagonal member 5-2
	8-1	0.73	Lower chord 4-1	0.70	Lower chord 4-1	0.70	Lower chord 4-1

Colored cells: ■ 10 or more members with value exceeding 1.0

■ 5 or more members with value exceeding 1.0

after the fracture of each member are shown in Table 4 and Fig. 8. Table 4 also shows the location of members which had the maximum verification value and the number of members whose verification value exceeded 1.0.

The verification values of the surrounding members tended to exceed 1.0 when diagonal members near the girder ends and the upper chord members fractured, and the number of members with verification value exceeding 1.0 also tended to increase. When the lower chord members fractured, verification values fell below 1.0 because the axial force before rupture was transmitted by the floor system, as mentioned above. When diagonal members fractured, it was found that the verification values increased because the bending moment increased in the surrounding members of the same main truss where the fracture occurred, and the axial force increased in the members of another main truss on the opposite side.

5.3 Influence of train load and corrosion on safety margin

An investigation focusing on the difference between the design load and the actual train load revealed that the verification values in Case 2 were smaller than those in Case 1 by about 10%, which is small compared to the reduction ratio derived from the ratio of the train load to the design load (1.29). The location of the maximum verification value and the number of members with a verification value exceeding 1.0 differed slightly between these two cases but the difference in verification values was small.

Furthermore, regarding the presence or absence of corrosion in the panel points, in Case 3, the capacity of the panel points decreased and panel point fractures occurred before that of the truss member in some cases, despite there being little difference in the verification values. When panel points fracture, there is a greater likelihood of this leading to a bridge collapse than the fracture of other

members because many truss members are connected at panel points. However, although the thickness of the panel points was assumed to be half the original thickness due to corrosion in this study, the maximum panel point verification values were 1.06. This demonstrated the possibility of knock-on fractures occurring in panel points was relatively low so long as the degree of corrosion is less than this assumption.

5.4 Extraction of potential fracture critical members (FCM)

Figure 9 shows a color coded map representing maximum verification values for surrounding members after fracture of a member according to their thickness. The figure shows that the highest surrounding member verification values occur when diagonal member 2 (second from the end) fractures, while the second largest values can be seen when diagonal members 1 and 4 break. Overall, verification values tend to increase when diagonal members near girder ends and upper chord members fracture. As such, these members can be regarded as fracture critical members (FCMs).

Fig. 9 Map of verification value (maximum verification value after the fracture of each member)

Figure 9 indicates that prioritizing maintenance of members with high verification values can be an effective countermeasure against bridge collapse.

6. Conclusion

In order to investigate safety margins before bridge collapse, the behavior of a standard existing railway steel through-truss bridge after the fracture of each truss member was studied using three-dimensional finite element analyses and members surrounding the fractured member were verified. The findings obtained in this study are shown below.

- (1) In the analysis of an open floor type steel truss bridge, the floor system was modelled using beam elements and shell elements. Results showed that both methods led to almost the same evaluation outcome for the safety margin of each member before bridge collapse.
- (2) When applying an actual train load instead of the design load, the verification values of the members surrounding the fractured member decrease. This fall however, is not as large as the ratio of the train load to the design load.
- (3) Regarding the corrosion of the panel points, in this study it was assumed that the thickness of the gusset plates was reduced uniformly by half due to corrosion. Although maximum verification values obtained in the presence and in the absence of corrosion did not differ significantly, there were differences in the sequence of fracturing, for example, cases where the panel points fractured before the truss members.
- (4) In terms of results obtained on the target bridge used in this study: the lowest safety margin was obtained when the fracture occurred in the second diagonal member from the girder end. Overall, the safety margin tended to be lower in the case of fractures in diagonal members near the girder

ends and the upper chord members. Those members were classed as fracture critical members (FCMs). Conversely, lower chord members tended to present sufficient safety margins.

References

- [1] Commission of U.S. Minneapolis Bridge Collapse Accident Technical Survey: *Report of U.S. Minneapolis Bridge Collapse Accident Technical Survey*, Ministry of Land, Infrastructure and Transport of Japan, 2007 (in Japanese).
- [2] T., Tamakoshi: "Actions regarding evaluation of redundancy in road bridge design," *Proceedings of 17th Symposium on Steel Structures and Bridges*, Japan Society of Civil Engineers, pp.1-14, 2014 (in Japanese).
- [3] H., Nagatani, N., Akashi, T., Matsuda, M., Yasuda, H., Ishii, M., Miyamori, Y., Obata., H., Hirayama and Y., Okui : "Structural redundancy analysis for steel truss bridges in japan," *Doboku Gakkai Ronbunshuu A*, Japan Society of Civil Engineers, Vol.65, No.2, pp.410-425, 2009 (in Japanese).
- [4] Committee for Redundancy of Steel Structures: *Guideline for redundancy evaluation*, Japan Society of Civil Engineers, 2014 (in Japanese).
- [5] M., Saito, M. Ikeda, F., Tajima and K., Yui: "Redundancy analysis of through truss bridge with open-type floor system," *Proceedings 71st Annual Meeting*, Japan Society of Civil Engineers, 2016 (in Japanese).
- [6] Y., Goto, N., Kawanishi and I., Honda: "On impacts caused by sudden failure of diagonal tension member in steel truss bridges," *Kozo Kogaku Ronbunshu. A (Journal of Structural Engineering. A)*, Japan Society of Civil Engineers, Vol.56A, pp.792-805, 2010 (in Japanese).
- [7] URS Corporation: "Fatigue evaluation and redundancy analysis, Bridge No.9340, I-35W over Mississippi river," *Draft report*, 2006.

Authors

Masamichi SAITO, Dr. Eng.
Steel & Hybrid Structures Laboratory,
Structures Technology Division
Research Areas: Hybrid Structures, Steel Structures

Manabu IKEDA, Dr. Eng.
Steel & Hybrid Structures Laboratory,
Structures Technology Division (Former)
Research Areas: Hybrid Structures, Steel Structures