DAMAGE ANALYSIS OF BRIDGE AFFECTED BY TSUNAMI IN THE GREAT EAST JAPAN EARTHQUAKE

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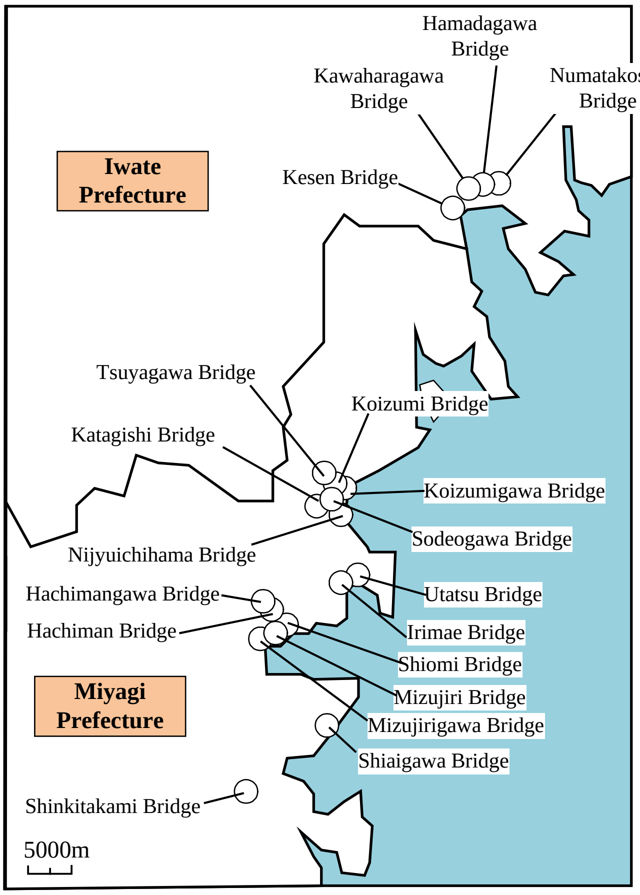
Abstract

A tsunami generated by the Great East Japan Earthquake hit the coastlines of East Japan. We evaluated outflow conditions for 37 bridges by β (the ratio to resistance and tsunami force). β was found to be effective in judging the girder outflow. Based on video analysis, the tsunami height and velocity variations in Shizugawa can be estimated. Besides, two impact mechanisms on the bridge girder at Shizugawa were analyzed (First, as the tsunami actually acted on the girders without submerging the bridge, ; the second for the tsunami totally submerging the girder). It is noted that for both the mechanisms, the girder is strong enough to resist tsunami.

Introduction

The 2011 Tohoku earthquake, also known as the 2011 Great East Japan



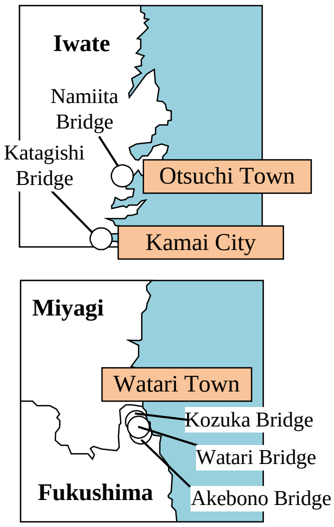


Fig.1 General Tohoku area

Table 1 Definition for damage ranks

Damage level	Outflow condition of superstructure
Rank A	Flowed out compeletly
Rank B	Moved but not divorced from abutment
Rank C	Slight damage

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Earthquake, was a magnitude 9.0 undersea earthquake that occurred at 14:46 (JST) on 11th March 2011, with its epicenter about 130 km southeast of Oshika Peninsula. The tremendous tsunami triggered by the earthquake inflicted tremendous destruction throughout East Japan. According to the reports of the Japan Meteorological Agency, inundation heights of 7 to 12 meters were observed in the northern parts of Fukushima Prefecture to the southern parts of Iwate Prefecture.

Immediately following the great Earthquake, the authors conducted several field investigations in the disaster zones of Japan. As shown in **Fig.1**, 24 bridges (variously made of 37 different types of girders) with their positions near the coastline will be studied in this paper. Firstly, we conducted evaluations of girder outflow by β value (ratio to girder resistance and tsunami lateral force). Secondly, discussion of the drag coefficient which is one of the parameters deciding tsunami force, will be conducted.

In one of the most severely damaged areas, the study selected Shizugawa Town is selected as an example for the deep analysis of tsunami. The tsunami height and velocity histories were estimated based on videos records of the tsunami process. Based on the obtained tsunami characteristics, two different mechanisms (firstly, where the tsunami just acted on the girders without submerging them; the other where the tsunami totally submerged girders) on the bridge girders (Hachiman Bridge) are summarized and the outflow analysis of girder is conducted.

<u>Evaluation of Bridge Outflow by β Value</u>

In this chapter, the authors first estimated the tsunami's lateral force on girder along with girder lateral resistance. After that, the β, i.e., the ratio of girder lateral resistance and tsunami lateral force, is computed to ascertain the relationship between β and girder outflow conditions due to tsunami. As a result, the levels of β for Rank A and C are quite different.

Secondly, the reasons why the β of some bridges do not coincide with their actual damage conditions are discussed. Before computing β, the outflow conditions of girder are defined as illustrated in **Table 1**.

Tsunami force can be calculated by Eq. 1, while girder lateral resistance can be computed by Eq. 2 and Eq. 3. When bridge girder is just acted upon by tsunami and not submerged, Eq. 2 is used and buoyancy should be ignored. When bridge girders are submerged totally by wave, Eq. 3 is applied and buoyancy should be considered (Eq. 2 and Eq. 3 are discussed detailed in following content).

$$F = \frac{1}{2}\rho_w C_d v^2 A_h \tag{1}$$

$$S = \mu W \tag{2}$$

$$S = \mu(W - U) \tag{3}$$

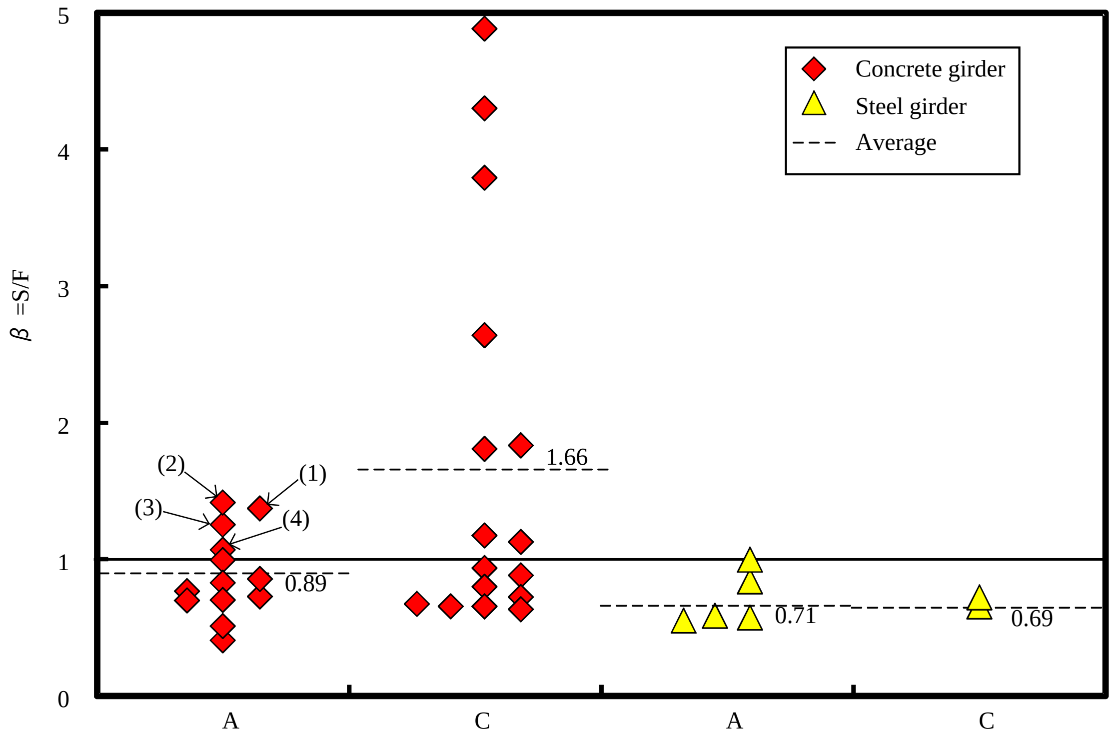

Fig.2 β ratios for entire Tohoku

Where, ρ_w is density of water (1030kg/m^3); C_d is the drag coefficient with its value decided based on references (Japan Road Association, 2002); v is tsunami velocity (6.0m/s is used based on surveys of the Tohoku area); A_h is the projected pressure area of the girder in the horizontal direction; μ is friction coefficient (0.6, based on research of (Rabbat, 1985)); W is the girder's dead load , and U is buoyancy as computed by Eq. 4:

$$U = \rho_w g V \tag{4}$$

Where, V is volume of the bridge girder inner the tsunami. Thus, the indicator β is computed by Eq. 5:

$$\beta = \frac{S}{F} \tag{5}$$

If β is smaller (greater) than 1.0, this indicates girder resistance is smaller (greater) than the tsunami force, which means girder is easy (difficult) to flow out. For the tsunami velocity (v) in Eq. 1, based on many recorded videos from throughout the Tohoku area, the average value of 6.0m/s (Li, 2012) is applied.

Fig. 2 illustrates the computed β for total 37 different types of girders. When β is larger than 1.5, the girders suffered Rank C damage; When β is smaller than 0.6, the

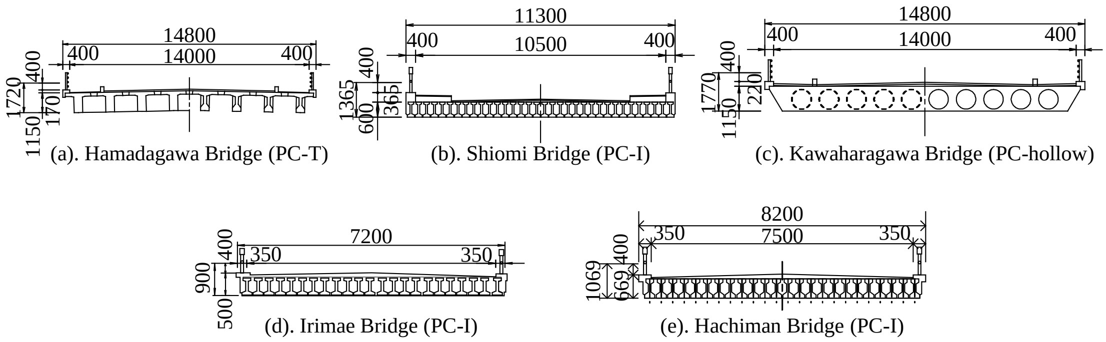

Fig.3 Section view of bridges with β ratios greater than 2.0

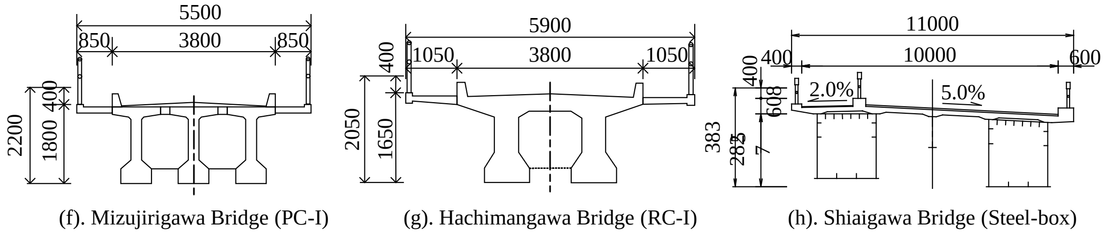

Fig.4 Section view of bridges of β ratios smaller than 0.6

girders suffered the damage of Rank A. Thus, it is concluded that a gap of β value exists between Rank A and Rank C, and it is effective to apply β evaluating outflow of girder generally.

For checking the characteristics of easily outflowed and difficultly outflowed bridges, the section views of difficultly outflowed bridges with β greater than 2.0 are shown in **Fig.3**. While the section views of easily outflowed bridges with β smaller than 0.60 are illustrated in **Fig.4**. It is found that the girders of difficultly outflowed bridges all belong to the PC (pre-stressed concrete) type. Three of 5 girders are made by PC-I. Further, it is discovered that the difficultly outflowed girders are relatively in flat shape. As to the difficultly outflowed bridges, overall girder height is in small level and girder length is in great level compared with those for the easily outflowed bridges.

If the bridge height is small, the tsunami impacting area per unit girder length has the tendency to be small. While, if the bridge width is greater, the girder weight per unit length will have the tendency to be greater. Thus, greater β values occur. As a result, the authors consider that greater bridge widths and shorter bridge heights cause the bridges to have greater β values, leading to greater resistant capability.

In previous research (Kenji, 2010; Yulong, 2011), obvious β level difference was found for Rank A and Rank C. But in Fig.2, some β of Rank A concrete girders are larger than 1.0 and some β of Rank C girders are smaller than 1.0.

As for the reason, firstly the authors note that tsunami velocity affects tsunami

Table 2 Rank A concrete bridges with β greater than 1.0

Rank A concrete bridges with β greater than 1.0			
No.	Name	β	Type
(1)	Numatakosen	1.37	PC-T
(2)		1.41	PC-T
(3)	Shimoya	1.25	PC-T
(4)	Kozuka	1.07	PC-T

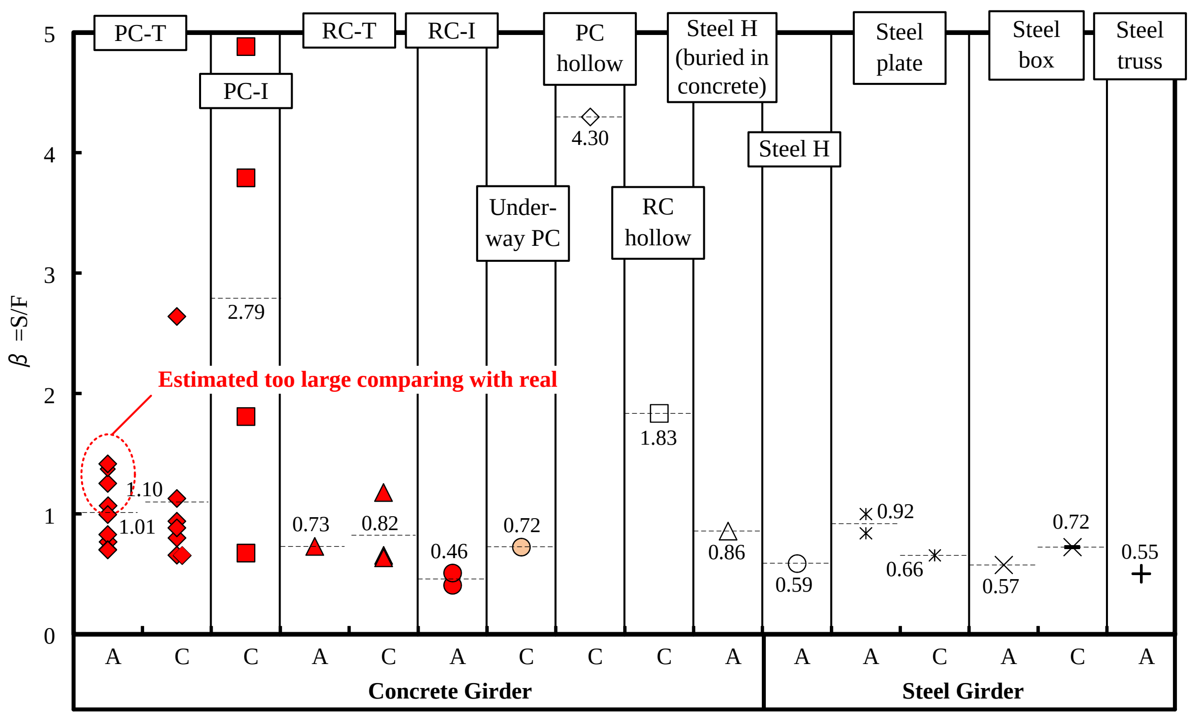

Fig.5 Classification of β by different type

force significantly. At the moment, the velocity 6.0m/s is applied, the velocities in different areas might be different. Therefore, the authors would like to adopt a simulation method to modify the velocities area by area, in order to obtain more accurate β values.

Table 2 shows the bridge details, the β of which belong to the above situations. It is found that all the Rank A girders having larger β than 1.0, belong to PC-T types. Thus, firstly the β are classified by type, as shown in **Fig.5**, to check the characteristics of PC-T. The authors found that compared with the β of other concrete girders for Rank A, the PC-T has great β with the average as 1.01. Therefore, the authors suppose that the tsunami forces on PC-T girders are estimated smaller than actual, which therefore yielded greater β than actual.

Considering the reasons why the tsunami forces are evaluated too low other than due to the effects of tsunami velocity, associating Eq. 1, the drag coefficient C_d is considered to be estimated too small. When it comes to the previous calculation method

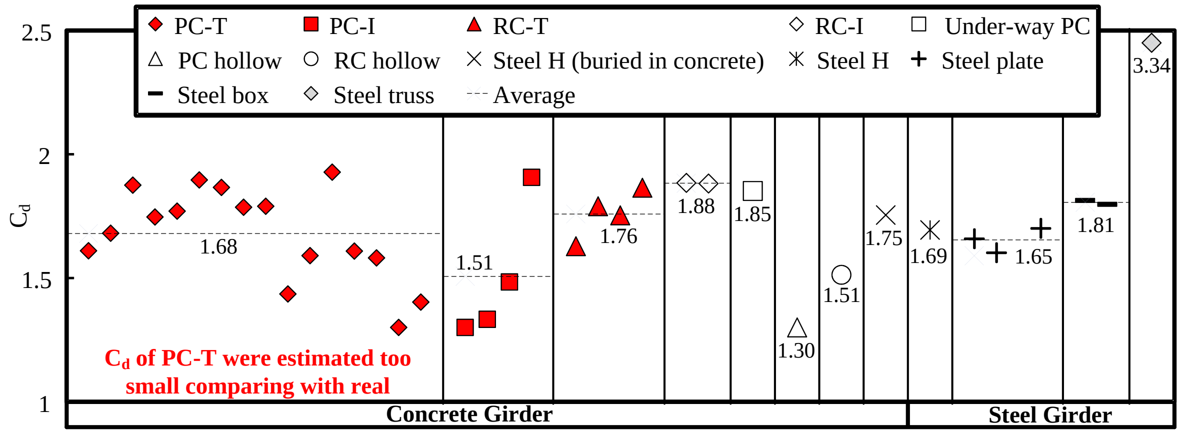

Fig.6 Influence from different girder types

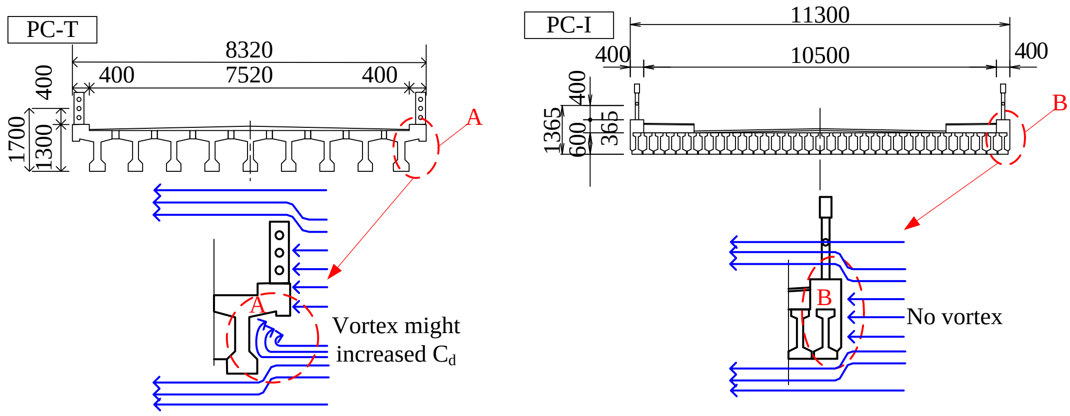

Fig.7 Reason why drag coefficient of PC-T deck might be estimated too small

of drag coefficient, based on the specification (Japan Road Association, 2002) this can be computed by Eq. 6 and Eq. 7 (Eq. 7 is for calculation of bridge with truss type, while Eq. 6 is for other types):

$$C_d = \begin{cases} 2.1 - 0.1(B/D), & 1 \le B/D < 8 \\ 1.3, & 8 \le B/D \end{cases} \tag{6}$$

$$C_d = 1.35/\sqrt{\phi} \quad (0.1 \le \phi \le 0.6) \tag{7}$$

Where, C_d is the drag coefficient; B is the bridge width (m); D is the bridge height (m); Φ is the fill rate as the ratio to the area of truss and the area of external contour for the truss.

In order to plot the characteristics of drag coefficient distribution of PC-T, the drag coefficient distributions are classified based on type as well, as shown in **Fig.6**. From **Fig.6**, it is found that the average drag coefficient of PC-T is closer to other concrete types. However, considering the real situation, as shown in **Fig.7**, which gives wave flow and action patterns on PC-T and PC-I girders of similar scales, due to the vortex phenomenon at the groove part of girder, the impact force on PC-T can be increased, which can lead up to relatively larger drag coefficients compared to PC-I girder. Therefore, by using same equation to compute drag coefficient, the drag coefficient of PC-T were possibly estimated too low .

After that, it is known that the β of steel girders are smaller than 1.0 and this indicates weak tsunami resistance, as a result of small girder weights compared to concrete girders.

In summary, if applying the velocity 6.0m/s, girders with β greater than 1.5 suffered Rank C damage and β of girders suffering Rank A damage are all smaller than 1.5. Thus, girders with β value greater than 1.5 are considered to be safe.

TSUNAMI CHARACTERISTICS IN SHIZUGAWA TOWN

After the tsunami strike, many areas of East Japan suffered great damage. In this chapter, Shizugawa Town, located at Minamisanriku township of Miyagi Prefecture is selected as an example to more deeply analyze the tsunami characteristics.

Tsunami height and velocity histories are significant to any analysis of tsunami damage. Thus, firstly the estimation of tsunami height and velocity variations of Shizugawa Town, based on with the application of videos recording the tsunami process, will be discussed. After that, the relationship between tsunami height and velocity variations will be studied. The results obtained in this chapter will be the basis for the outflow evaluation of bridge girder in the next chapter.

After collection, two videos could be used to estimate tsunami height and

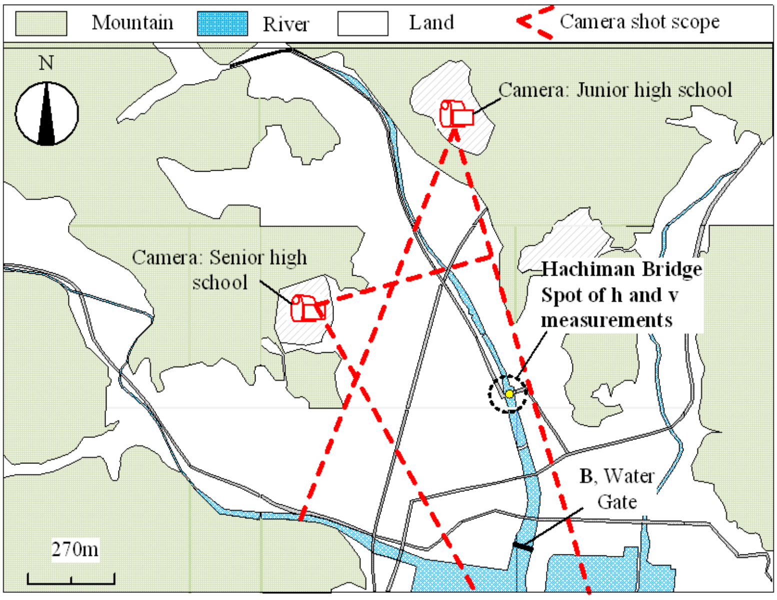

Fig.8 Study Area

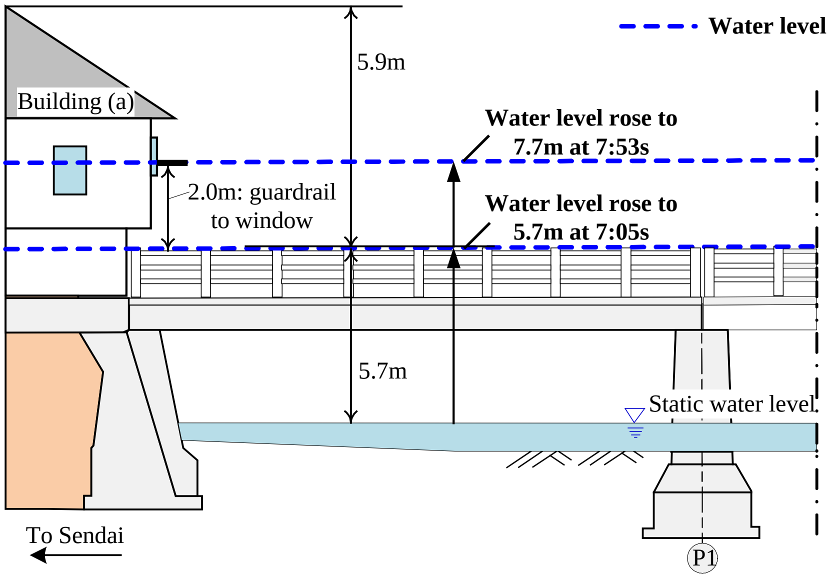

Fig.9 Side View of Hachiman River Bridge

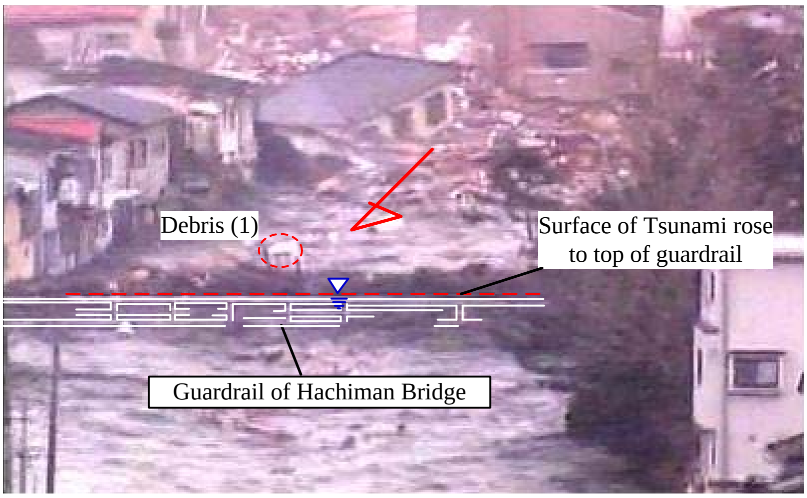

Fig.10 Tsunami wave at 7:05s

velocity variations, and the shot scope is shown in **Fig.8**. The estimation of tsunami height and velocity were done at the location near the Hachiman Bridge.

In **Fig.9**, firstly the method about how to measure tsunami height is introduced. Before measurement, some reference heights could be estimated refer to the length of girder span (11.98m): the height from the top of guardrail to the static water level is estimated as 5.7 m; the height from the top of building (a) to the top of guardrail is estimated as 5.9 m; the height from top of guardrail to the window of building-(a) is about 2.0 m. Then during the tsunami process, in video time, at 7:05s, the tsunami reached the top of bridge guardrail, from which we know the tsunami height was 5.7 m. After that when time passed to 7:53s, the tsunami reached the window of the building (a) and it is known that the tsunami height at this time was about 7.7 m.

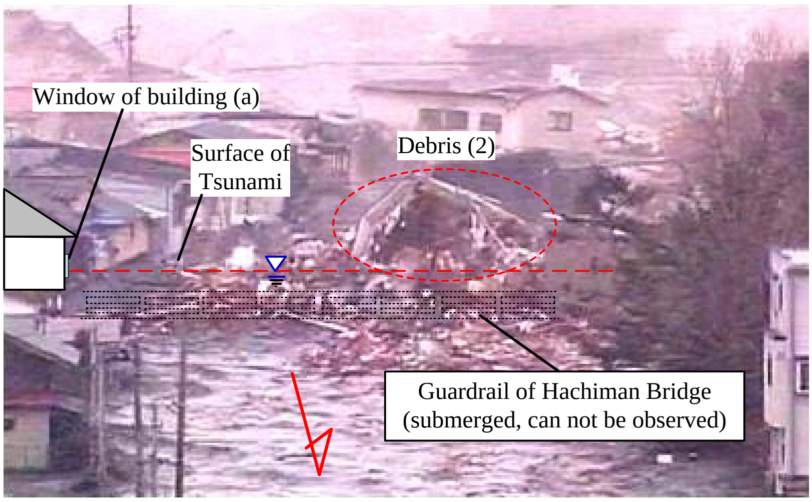

Fig. 11 Tsunami wave at 7:53s

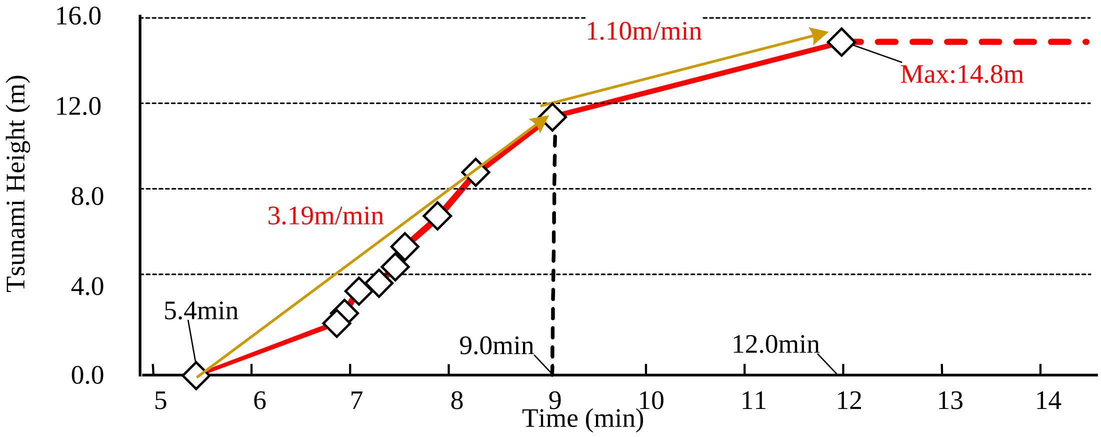

Fig.12 Time-depended Variation of Tsunami Height (Hachiman Bridge)

Besides, the corresponding video screen of measurements at 7:05 and 7:53s are plotted in **Fig.10** and **Fig.11**.

Afterwards, by using the same method, more tsunami heights at different time points at Hachiman Bridge were estimated and summarized in **Fig.12**. At 5.4 min., the tsunami hit Hachiman Bridge. Before 9.0min., the tsunami height rose with the speed of 3.19 m/min. With the passage of time, the tsunami height ran up to the maximum 14.8m at 12.0min., at a pace of 1.1 m/min. The average rising speed of tsunami height is estimated as 2.24 m/min. Tsunami height was rising in relatively small pace, inferring that the wave shape was not the bore type.

Further, investigations of the relationship between the tsunami height and the tsunami velocity will be shown in the following contents. As presented in **Fig.10** and **Fig.11**, much debris was found flowing through the Hachiman Bridge, such as the debris (1) and debris (2). For the flow process of one piece of debris, two distinguished spots that the debris flowed through could be found. The distance between the two

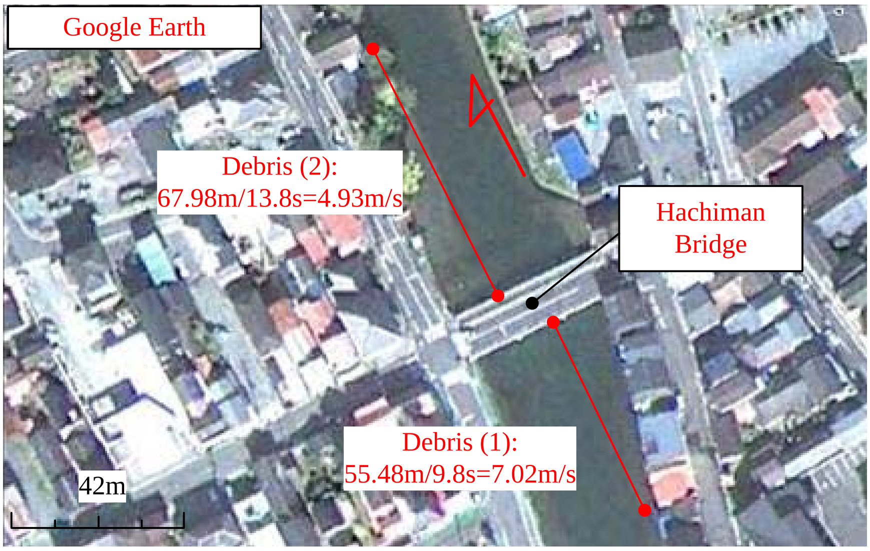

Fig.13 Estimation of Tsunami Velocity

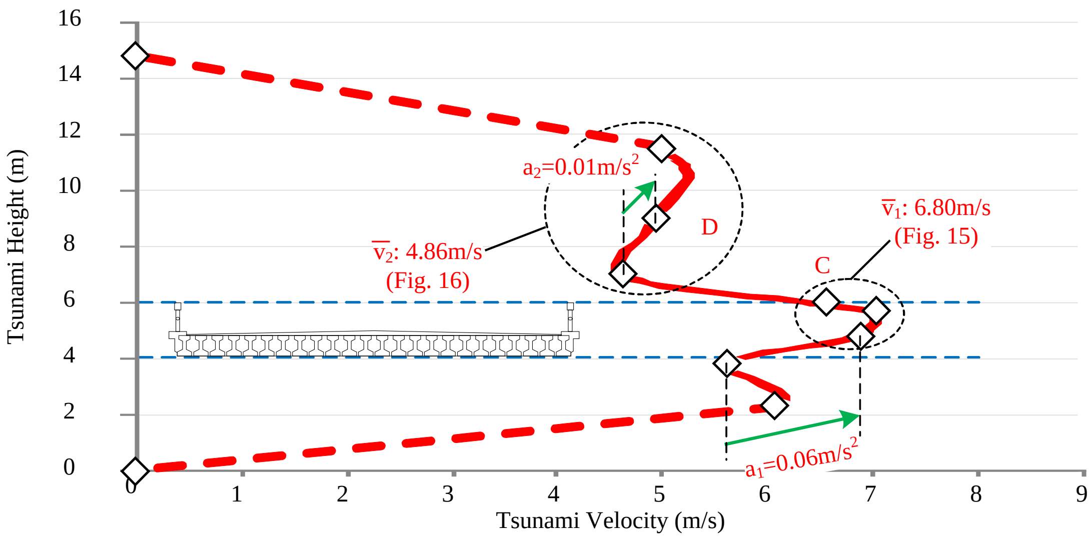

Fig.14 Relation between v-h (Hachiman Bridge)

spots can be measured using the functions of Google Earth. The time for the debris flowing through the two spots can be obtained by stopwatch. Finally, the tsunami velocity is estimated by the ratio to the distance and the time span. As shown in **Fig. 13**, the measurements with the use of debris (1) and debris (2) are shown to explain how to get tsunami velocity. Based on same method, velocities at different time points have been estimated, where are close to Hachiman Bridge.

After the wave height and velocity variations were estimated, the relationship between tsunami height and tsunami velocity is obtained and shown in **Fig.14**. Herein, we treat the velocity as 0 when tsunami height is 0. When the tsunami began to fall back, the velocity was treated as 0 as well. Before the tsunami reached about 2m, the tsunami velocity rose to about 6 m/s. After a slight decrease, the velocity increased continually before the tsunami hit the bridge girder. The maximum velocity of 7.02m/s occurred when the tsunami was attacking the bridge girder. After the tsunami ran up to about 6m, the bridge was inundated and the tsunami velocity began to decline to became about 4.86m/s (average of area D in **Fig.14**). After tsunami height rose to about 12m, velocity decreased to 0 gradually.

From **Fig.14**, we see two characteristics. First is that the tsunami velocity continued to increase generally to its maximum before the tsunami began to overflow the bridge girder. The second is that the tsunami velocity decreased suddenly when the bridge girder was inundated. With respect to the two characteristics, the following is obtained. With the girder top of Hachiman Bridge is the same level as the embankment top, thus when tsunami rose to the level of the girder top, it would overflow to the land area and that led to the decreases in tsunami energy and velocity.

ESTIMATION OF GIRDER OUTFLOW OF HACHIMAN BRIDGE

In this chapter, the authors will discuss the outflow mechanisms of Hachiman Bridge in detail, based on the tsunami height and velocity variations. Hachiman Bridge, the location of which is shown in **Fig.8**, survived during the tsunami impact. The sectional view of the bridge can refer to **Fig.16**. The bridge girders belong to the PC-I type with the width as 8.2m, height as 1.069m.

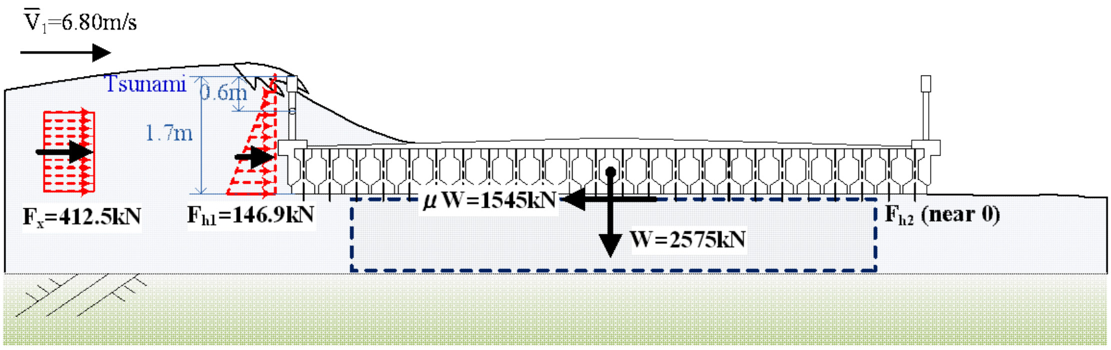

Fig.15 Mechanism 1 (tsunami just impact on bridge girder)

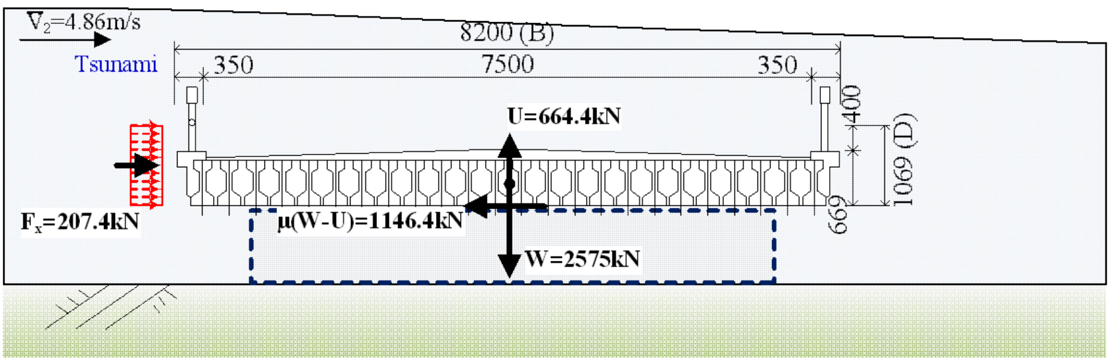

Fig.16 Wave shape 2 (bridge girder is entirely inundated)

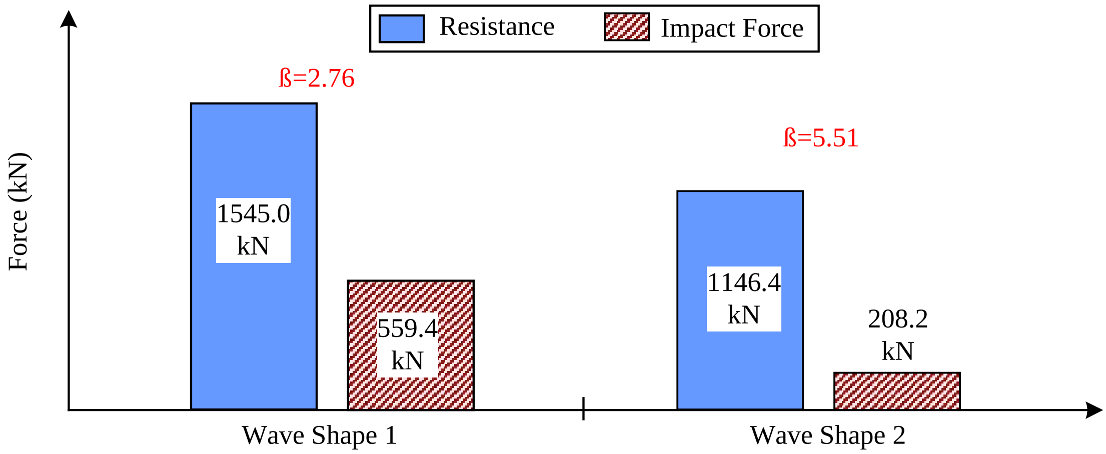

Fig.17 Calculation of β in Two Wave Shapes

From the tsunami height variation at Hachiman Bridge, as was concluded in previous chapter, the authors propose two kinds of mechanisms at the time points when tsunami just acted on girder and when the girder was totally submerged. The images of the two mechanisms are shown in **Fig.15** and **Fig.16**.

As shown in **Fig.15**, Mechanism 1 illustrates the condition when tsunami just impacted the girder. From the actual tsunami condition presented in **Fig.10**, the tsunami surface at the impacting side was close to the top of bridge guardrail. However, tsunami at the upstream side just reached the bottom of girder. The difference of tsunami height is about 1.7m. Therefore, the difference of water head is 1.7m. The Mechanism 1 corresponds to the C area in **Fig.14**, in which the average velocity is estimated as 6.80m/s.

Secondly, Mechanism 2 is presented in **Fig.16**, when the bridge girder was inundated entirely. The tsunami surface at this time is relatively in flat shape without great difference of water head between the impacting side and the upstream side. The Mechanism 2 corresponds to the area D in **Fig.14**, where the average velocity is estimated as 4.86m/s.

For the two different mechanisms, the girder outflow by evaluating the β to girder resistance and tsunami force will be discussed. For Mechanism 1 (**Fig.15**), the tsunami force is considered to be composed by two parts (hydrodynamic force and hydrostatic force (Komatsu, 2011)), and can be computed by Eq. 8:

$$F = \frac{1}{2}\rho_w C_d A v^2 + C_m \rho_w AB\frac{dv}{dt} + (\rho_w g h_1 A_1 - \rho_w g h_2 A_2) \tag{8}$$

Where, F is tsunami force; ρ_w is density of water (1030kg/m^3); C_d is drag coefficient, which is calculated as 1.33 based on Eq. 6; v is tsunami velocity (m/s); A is projected pressure area of girder in horizontal direction (m^2); C_m is inertia coefficient, which is assumed to be 1.0; B is bridge width (m); dv/dt is acceleration of tsunami velocity (m/s^2); h_1 and A_1 is the water depth and pressure area at the downstream side of girder; h_2 and A_2 is the water depth and pressure area at the upstream side of girder.

Firstly, the hydrodynamic force is evaluated (first and second items at right side of Eq. 8). As shown in **Fig.14**, average velocity 6.80m/s and the greatest acceleration a_1 ($0.06m/s^2$) of area C is adopted. Thus, the hydrodynamic force is calculated as 412.5kN. Secondly, the hydrostatic force is computed (third item at right side of Eq. 8). As presented in **Fig.15**, tsunami at the impacting side of girder is assumed to just attach the top of guardrail. The hydrostatic force is computed as 412.5kN. For the upstream side of girder, as the tsunami was just attaching the bottom of bridge girder, hydrostatic force is ignored. Thus, the hydrostatic force is calculated as 146.9kN. The hydrodynamic force (412.5kN) is about 2.81 times the hydrostatic force (146.9kN). This is the reason why in first chapter, when computing tsunami force, the authors only considered hydrodynamic force (Eq. 1). Further, the resistance of the girder and the β can be computed by the former equations. For Mechanism 1, the resistance is computed as 1545kN which is 2.76 (β) times larger than tsunami force (559.4kN).

With respect to Mechanism 2, the difference of water head is ignored and only the hydrodynamic force is considered. Further, as the girder is entirely inundated, the buoyancy should be considered when computing girder resistance (Eq. 3). From **Fig. 14**, the average velocity 4.86m/s and greatest acceleration $0.01m/s^2$ in area D are used for the computation of tsunami force. The resistance is computed as 1146.4kN, which is 5.51 times (β) greater than tsunami force (208.2kN).

In summary, as illustrated in **Fig.17**, β of both two mechanisms show the large levels (2.76 and 5.52), which explains the survival of girder.

Conclusions

(1) From the outflow evaluation of 37 bridge girders, it is summarized that if applying the velocity 6.0m/s, girders with β greater than 1.5 suffered Rank C damage and girders with β smaller than 0.5 suffering Rank A damage. Thus, it is possible to use β to evaluate outflow of girders.

(2) As a key reason why the β of PC-T girders are larger than other concrete girder types, it is noted that the drag coefficient of PC-T is estimated at less than actual, because during the computation of drag coefficient, the vortex effect is not considered.

(3) Based on the study of variation for tsunami height in Shizugawa Town, the average rising speed at Hachiman Bridge was 2.24 m/min. Tsunami height rose at a slower pace, which inferred the wave shape was not the bore type as often happens

(4) Based on the tsunami height variation at Hachiman Bridge of Shizugawa Town, two mechanisms on the bridge girder at Shizugawa are analyzed. One type is that tsunami just acted on girder and did not submerge it (the buoyancy is ignored); the

other type is that tsunami submerged girder totally (the buoyancy is considered). It is concluded that for both the two mechanisms, β of Hachiman Bridge are 2.76 and 5.51, respectively. Therefore, the girder resistances of Hachiman Bridge are great enough to resist tsunami action.

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