EVALUATION OF STRENGTH OF EXISTING BEARING SUPPORTS IN BRIDGES SUBJECTED TO TSUNAMI-INDUCED FORCE

Hisashi NAKAO1, Toru SUMIMURA2, Yoshihiro MORIYA3 and Jun-ichi HOSHIKUMA4

1Member of JSCE, Research Specialist, Center for Advanced Engineering Structural Assessment and Research (CAESAR), Public Works Research Institute (Minamihara 1-6, Tsukuba, Ibaraki 305-8516, Japan)
E-mail: nakao55@pwri.go.jp
2Member of JSCE, Dept. of Eng., Kawakin Core-Tech Co., Ltd.
(Umeda Kita Place, Shihata 1-14-8, Kita-ku, Osaka 530-0012, Japan)
(Former Collaborating Researcher, CAESAR, Public Works Research Institute)
E-mail: sumimura@kawakinkk.co.jp
3Member of JSCE, Dept. of Eng., Kawakin Core-Tech Co., Ltd.
(Kawaguchi 2-2-7, Kawaguchi, Saitama 332-0015, Japan)
(Former Collaborating Researcher, CAESAR, Public Works Research Institute)
E-mail: y-moriya@kawakinkk.co.jp
4Member of JSCE, Head, Kumamoto Earthquake Recovery Division, National Institute for Land and Infrastructure Management
(Kawayo 3574, Ooaza, Minamiisao, Aso, Kumamoto 869-1404, Japan)
(Former Chief Researcher, CAESAR, Public Works Research Institute)
E-mail: hoshikuma-j92ta@mlit.go.jp

This paper presents a method for evaluating the strength of bearings supports in bridges subjected to tsunami-induced forces, to allow an estimation of the effects of a tsunami on bridge functionality. The tsunami-induced forces striking the bearing supports were studied in previous flume tests and numerical analyses conducted by the authors. Through comparison with the actual performance of bridges affected by the tsunami in the 2011 Great East Japan Earthquake, we verified the effectiveness of the method in estimating the tsunami effects on bridges.

Key Words : tsunami effect, bridge, bearing support, strength of bearing support

1. INTRODUCTION

Many bridges in and around the coastal regions of Tohoku were washed away by tsunami forces in the 2011 Great East Japan Earthquake1). The recovery of the functionality and viability of bridges for emergency routing was delayed due to the washed-away superstructures. As evident from experiences in the earlier Tokai, Tonankai, and Nankai earthquakes, a tsunami was highly likely to damage bridge functioning throughout the coastal zones, thus making reliable evaluation of the tsunami-induced effect on bridges and early recovery methods of bridge functions essential and invaluable metrics.

Earlier studies have used flume tests2)-6) and numerical analyses7)-10) to study the impact of tsunami on bridges, to identify the tsunami-induced forces and measures to mitigate impacts. The tsunami properties and the acting mechanisms affecting the superstructures were also studied using the tsunami video of the bridges11)-13). These studies focused mainly on the tsunami properties and the tsunami-induced force on the superstructures. However, studies focusing on the perspective of damage modes and mechanisms of bearing supports are few despite the fact that many washed-away bridges involved failure of bearing supports14). In previous studies, assessments of washed-away bridges were performed by comparing the bridge resistance calculated simply based on the superstructure weight vis-a-vis tsunami-induced force. However, more precise methods of evaluating the resistance are essential for evolving methods to determine the degree of impact affecting bridge functioning, notably on the impact side15).

Bearing support designs are based on the Road Bridge Bearing Manual16) and on the Standard De-
sign for Highway Bridges Bearings\textsuperscript{17}). Bearing supports are designed such that the stress as calculated using the stress-calculation method from the Road Bridge Bearing Manual\textsuperscript{16} and Standard Design for Highway Bridges Bearings\textsuperscript{17}, satisfies the allowable stress requirements for the materials. On the other hand, according to the results of loading tests on the existing steel bearings, the final destruction often meaningfully differs from the calculated prediction. Thus presently, there is no established method with highly reliable accuracy for strength evaluations\textsuperscript{14},\textsuperscript{18},\textsuperscript{19}. Overseas, few studies evaluate steel bearings in relationship to earthquake or tsunami\textsuperscript{20}. Studies that quantitatively evaluate the reaction force due to the tsunami are also few. Therefore, to quantitatively evaluate the degree of effect tsunami might have on existing bridges, the strength of bearing supports, and the tsunami-induced force have to be evaluated with a greater precision.

This paper demonstrates how the damage mode of bearing supports due to the tsunami can be classified using results of investigations of failure of bearing supports from washed-away bridges to yield a high-accuracy method for evaluating bearing supports.

Next, the tsunami-induced forces striking the superstructures were based on flume tests\textsuperscript{23},\textsuperscript{25} and numerical analysis, and an evaluation method for the reaction force upon bearing supports was suggested.

Finally, we tested how our proposed method could be applied to study a bridge actually affected by the tsunami, to verify its usefulness through comparison with actual disaster results.

### 2. DAMAGE MODE AT BEARING SUPPORTS

(1) Pot bearing (BP bearing)

Figure 1 shows the damage on pot bearings due to tsunami. The primary damage mode for pot bearings is failure of the anchor bolt, set bolt, and/or side block bolt. In the case of the failure of the anchor bolt in a straight state, the anchor bolt is fractured by vertical force. On the other hand, the trail on the pier shows the anchor bolt is broken by a horizontal force. The damage to set bolts and side block bolts is considered to have come from the lift of the superstructure by the vertical uplift force because the upper shoe protrusion, which resisted the transverse direction force, is unbroken.

(2) Line bearing

The damage mode of the line bearings is shown in Fig. 2. Damaged portions can be seen at the lower shoes, anchor bolts, and pinch plates. From the loading tests of the line bearings\textsuperscript{19}, the main cause of the breaking of the lower shoe is thought to be the transverse direction force. Also, the main causes of the slipout of anchor bolts, and the bending deformation of anchor bolts and pinch plates are thought to be due to vertical forces.

(3) Rubber bearing

Figure 3 shows the damage mode of rubber bearings. From the investigation of rubber bearings\textsuperscript{26}, the failure of set bolts is confirmed to be due to vertical force. The anchor bolts failures are shown to be from the vertical force (at the end of pier) and the transverse direction force (at the center pier). However, the failure of the rubber bearings themselves is not known with certainty to have been caused by either the vertical or transverse forces because the tensile strength of rubber cannot be pre-
3. STRENGTH EVALUATION OF BEARING SUPPORTS

(1) Target bearing structure

This study focused on the pot bearings, rubber bearings, and anchor bars. We noted that members installed as parts of the joint surface of set bolts and anchor bolts, etc., should be designed to consider the effects of the simultaneous action of horizontal and vertical forces because the joint surfaces between the bearing, the superstructure, and substructure are affected simultaneously. However, based on the investigation of damage to bridges\textsuperscript{26,27}, the authors hold the primary cause of the broken bearings to be unidirectional force and thus, this study considered the horizontal and vertical impacts separately.

(2) Strength evaluation in transverse direction

The strength evaluation method of bearing supports in the transverse direction was suggested based on the damage mode of bearing supports in the past huge earthquake and the 2011 Great East Japan Earthquake tsunami.

In this study, the strength of bearing supports was evaluated not by the allowable stress using bearing supports design but by the ultimate tensile stress. The strength in the transverse direction was calculated as the horizontal force when the stress at the verification section reached the tensile stress based on the stress calculation method shown in the Road Bridge Bearing Manual\textsuperscript{16} and Standard Design for Highway Bridges Bearings\textsuperscript{17}. This method is considered advanced for evaluating with high accuracy, the strength of bearing supports against the tsunami. The transverse direction strength verification in each bearing structure targeted the parts shown in Fig. 4 based on damage mode due to the tsunami\textsuperscript{27} and the result of the loading test\textsuperscript{14}.

![Fig.2 Main damage modes of line bearing.](image)

![Fig.3 Main damage modes of rubber bearing.](image)
a) Anchor bolt (pot bearing, rubber bearing)

The strength of the anchor bolt in the transverse direction, assuming that anchor bolts are affected by uniform shear stress from horizontal force, is shown in Eq.(1):

\[ H_{u1} = A_s \times \tau_u \times n \]  

(1)

where, \( H_{u1} \) is the strength of the anchor bolt, \( A_s \) is the section area of the anchor bolt, and \( n \) is the number of anchor bolts. \( \tau_u \) is the ultimate shear stress of the anchor bolt, and calculated by Eq.(2), based on the Design Specifications for Highway Bridges.

\[ \tau_u = \sigma_u / \sqrt{3} \]  

(2)

where, \( \sigma_u \) is the ultimate tensile stress of the anchor bolt.

b) Connection between bearing and superstructure (pot bearing, rubber bearing)

The transmission of horizontal force between the bearing and superstructure is conducted through set bolts as shown in Figs.4(a) and 4(b), [2]-1 and the shear key as shown in Figs.4(a) and 4(b), [2]-2. To target bridges, there is the bearing structure combining these parts and thus, the strength of these parts was evaluated the large strength one of the strength at the set bolt \( H_{u21} \) or shear key \( H_{u22} \).

\[ H_{u2} = \max \left( H_{u21}, H_{u22} \right) \]  

(3)

where, \( H_{u2} \) is the strength in the transverse direction at the joint between the bearing and superstructure, \( H_{u21} \) is the strength of the set bolts, and is shown in Eq.(4) and Eq.(5):

\[ H_{u21} = A_s \times \tau_u \times n \]  

(4)

\[ \tau_u = \sigma_u / \sqrt{3} \]  

(5)

where, \( A_s \) is the section area of the set bolt, \( \sigma_u \) is the ultimate tensile stress of the set bolt, \( n \) is the number of set bolts, and \( \tau_u \) is the ultimate shear stress of the set bolt.

The strength of the shear key \( H_{u22} \) is shown in Eqs.(6)(7):

\[ H_{u22} = A_s \times \tau_y \]  

(6)

\[ \tau_y = \sigma_y / \sqrt{3} \]  

(7)

where, \( A_s \) is the section area of the shear key, \( \tau_y \) is the yield shear stress of the shear key, and \( \sigma_y \) is the yield stress of the shear key.

Interestingly, there is a possibility that the real strength of the connection between the set bolts and the shear key exceeds the calculated value because the shear key and the set bolts work simultaneously to resist the acting horizontal force.

c) Lower shoe-stopper (pot bearing)

The strength of the lower shoe-stopper was evaluated by the composition of the bending stress and shear stress as the cantilever beam shown in Fig.5.

\[ H_{u3} = \frac{\sigma_u \times a^2 \times b}{\sqrt{3}(12h^2 + a^2)} \]  

(8)

where, \( H_{u3} \) is the strength of the lower shoe-stopper, \( a \) and \( b \) are the sizes of the lower shoe-stoppers, \( h \) is the acting height of horizontal force, and \( \sigma_u \) is the ultimate tensile stress.
The acting height of the horizontal force in the actual design is set at the center of the connection height between the upper and lower shoes. However, in estimating the ultimate state of the bearings, we considered that the action position of the horizontal force was to the underside of the upper shoe because of the bending deformation of the lower shoe \(^{18}\) (Fig. 6). Therefore, this study considered the height from the verification-section point to the underside of the upper shoe to be the acting height \(h\).

d) Shear key on upper shoe bottom

Some of the existing, fixed pot bearings showed the horizontal force transmission position to be the continuous contact part between the circular shear key of the upper shoe bottom and the hole of the top of the lower shoe as shown in Fig. 4(a). In this case, the strength of this part \(H_{u4}\) was evaluated by the ultimate shear stress at the shear-key-joint of the underside of the upper shoe.

\[
H_{u4} = A_t \times \tau_u \\
\tau_u = \sigma_u / \sqrt{3}
\]

where, \(A_t\) is the section area of the horizontal direction, \(\tau_u\) is the ultimate shear stress of the upper shoe bottom, and \(\sigma_u\) is the ultimate tensile stress.

e) Rubber

The ultimate shear strain of the rubber bearing varies with shape, material, etc. The ultimate shear strain of the rubber bearing actually obtained was about 300% over the result of rubber bearing loading tests\(^{29}\). Therefore, the strength of the rubber \(H_{u5}\) in this study was calculated by Eq.(11) with the assumption that the ultimate shear strain was 300%.

\[
H_{u5} = A_e \times G_e \times \gamma_u
\]

where, \(A_e\) is the section area of rubber, \(G_e\) is the shearing modulus of the rubber, and \(\gamma_u\) is the ultimate shear strain (assumed to be 300%).

f) Anchor bar

This study includes bridges where the anchor bars handle the horizontal force transmission. Generally, the standard strength of the anchor bar is evaluated based on the shear strength of a section area of the anchor bar. However, for this study, the strength of the anchor bar \(H_{u6}\) was evaluated by Eq.(12) as per earlier research\(^{30}\).

\[
H_{u6} = \frac{1.7 \times A \times \sigma_u}{4 \times (d/\varphi + \beta)}
\]

where, \(A\) is the section area of the anchor bar, \(\sigma_u\) is the ultimate tensile stress of the anchor bar, \(d\) is the clearance between superstructure and substructure, \(\varphi\) is the diameter of the anchor bar, and \(\beta\) is the coefficient (=1.0)\(^{30}\).

(3) Strength evaluation in vertical direction

This section explains the strength evaluation used for bearing supports in the vertical direction. To more accurately evaluate the strength of bearing supports against the tsunami, the strength in the vertical direction was considered as the vertical force which, when applied, makes the checking system reach the ultimate tensile stress as the strength at the verification section was obtained based on the bearing loading tests\(^{14}\), as well as stress calculation methods in the Road Bridge Bearing Manual\(^{16}\) and the Standard Design for Highway Bridges Bearings\(^{17}\). The strength verification part in the vertical direction focused on the parts shown in Fig. 7 based on the damage mode and the findings of loading tests\(^{14}\).

a) Set bolt

The strength of the set bolt \(V_{u1}\) was evaluated using Eq. (13) assuming the vertical force acted equally on the set bolts.

\[
V_{u1} = A_s \times \sigma_{uw} \times n
\]

where, \(A_s\) is section area of the set bolt, \(\sigma_{uw}\) is the ultimate tensile stress of the set bolt, and \(n\) is the number of set bolts.

b) Side block bolt

As shown in Fig. 5, side bolts are set at the sides of the lower shoe to prevent separation between the upper and lower shoes from vertical uplift forces. Side block bolts are fixed L-type plates on the side of the lower shoe. The section X-X / Y-Y shown in Fig. 8 and side block bolts are verified at design time. The result of the pot bearing loading test\(^{14}\) showed that the weakest part of the pot bearing against the vertical uplift force was section X-X, but this weakest part was unbroken; it was the side block bolts that
were broken. Therefore, this study assumed the side block bolt was the weakest part, and its strength $V_{u2}$ was calculated using Eq.(14):

$$V_{u2} = \frac{2 \times \sigma_{tu} \times A_s \times n}{\sqrt{\left(\frac{L_1}{L_2}\right)^2 + 3}}$$  \hspace{1cm} (14)

where, $V_{u2}$ is the strength of the side block bolts, $A_s$ is the section area of the side block bolt, $\sigma_{tu}$ is the ultimate tensile stress of the side block bolt, $n$ is the number of side block bolts, and $L_1, L_2$ are the acting distances.

c) Anchor bolt

The strength of the anchor bolt $V_{u3}$ as calculated in Eq.(15) assumed the tensile stress at anchor bolts was acted upon equally by the vertical uplift force.

$$V_{u3} = A_s \times \sigma_{tu} \times n$$  \hspace{1cm} (15)

where, $A_s$ is the section area of the anchor bolt, $\sigma_{tu}$ is the ultimate tensile stress of the anchor bolt, and $n$ is the number of anchor bolts.

d) Rubber

The strength of the rubber $V_{u4}$ was calculated in Eq.(16) following the Manual for Highway Bridge Bearings\(^{16}\).

$$V_{u4} = A_r \times \sigma_{tu}$$  \hspace{1cm} (16)

where, $A_r$ is the section area of the rubber and $\sigma_{tu}$ is the ultimate tensile stress of the rubber.

(4) Evaluation of bearing strength in existing bridges affected by tsunami

a) Targeted bridges and materials used

The strength of bearing supports was calculated using the proposed method. This study targeted bridges for which there was detailed information on the bearing structure and the superstructure; the tsunami speed generated at the targeted bridge was obvious on the video records\(^{11,31)-33}\). That resulted in increasing the verifiable accuracy of the evaluation method. The targeted bridge bearing structure and bearing materials are shown in Table 1.

For the evaluation of bearing supports, the mechanical properties of the steel bearing supports were defined based on the characteristic value specified in JIS. In the Manual for Highway Bridge Bearings\(^{16}\), the minimum design stress for rubber is 2.0 N/mm\(^2\) in accord with loading tests. However, the actual ultimate tensile stress (at fracture) had the variance 3.0 N/mm\(^2\) to 5.0 N/mm\(^2\); thus this study evaluated the vertical strength of the rubber in the range of 3.0–5.0 N/mm\(^2\) of the tensile stress\(^{16}\).

b) Calculation result of bearing strength

Figure 9 shows the results of the strength of bearing supports in the transverse direction and Fig.10 shows the results of the strength of bearing supports in the vertical direction. The ultimate tensile stress of the rubber bearings of Kesen Bridge shown in Fig.10(e) was found to be in the range of 3.0–5.0 N/mm\(^2\) of the rubber tensile strength. Also, the values calculated by the design method (allowable stress) as specified in the Road Bridge Bearing Manual\(^{10}\) and Standard Design for Highway Bridges Bearings\(^{17}\) are shown in the figure. This paper relied on conventional design methods.

To find the weakest part, this study assumed that the bearing system broke at its weakest segment. Looking at the strength of the bearing in the transverse direction as shown in Fig.9, the ultimate stress of the bearings other than the anchor bar calculated using the suggested method exceeds the yield stress values obtained by conventional design methods calculations. The results indicated that the actual bearing ultimate stress exceeded the stress allowed in the Road Bridge Bearing Manual\(^{10}\) and Standard Design for Highway Bridges Bearings\(^{17}\). Also, there were relatively more broken anchor bolts than other links–meaning, the anchor bolts were the weakest parts.
Table 1 Bridges and materials used in the bearing support.

| Bridge name         | Koizumi Bridge | Yanoura Bridge | Shin-Aikawa Bridge | Kesen Bridge |
|---------------------|----------------|----------------|--------------------|--------------|
| Verification position | A1, A2, P1 | P1, P3 | P2, P4 | A1, A2, G1, P10, P2-G10 |
| Bearing style | Pot bearing | Pot bearing | Pot bearing | Pot bearing |
| SS400 | SS400 | SC450 | SS400 |
| SS400 | SS400 | SC450 | SS400 |
| SS400 | SS400 | SC450 | SS400 |
| SS400 | SS400 | SC450 | SS400 |
| SS400 | SS400 | SC450 | SS400 |
| SS400 | SS400 | SC450 | SS400 |
| SS400 | SS400 | SC450 | SS400 |
| SS400 | SS400 | SC450 | SS400 |
| SS400 | SS400 | SC450 | SS400 |
| SS400 | SS400 | SC450 | SS400 |
| SS400 | SS400 | SC450 | SS400 |
| SS400 | SS400 | SC450 | SS400 |

---: Not corresponding part

Verification section

1. Anchor bolt
2. Connection between Bearing and Superstructure
3. Lower Shoe-stopper
4. Shear Key of Upper Shoe Bottom
5. Rubber
6. Anchor Bar

Fig.9 Results of bearing strength calculation (transverse direction).
In many cases, the weakest part against vertical forces was the side block bolts. The next weakest part was the set bolts shown in Fig.10. On the other hand, the strength of the anchor bolt was relatively greater than those of the other parts. Therefore, where the anchor bolt was broken, we considered the cause could be the horizontal force. In the rubber bearings, the weakest part was different according to position (A1, A2 abutments and P1, P2, P3 piers: rubber or anchor bolt, P4 pier: rubber or set bolt) because of the variance in tensile stress values.

c) Comparison with failure of bearing supports due to tsunami

The bearing strength calculated in accord with the suggested method was compared with the damage state of the bearings following the tsunami. As an example, the damage state of the steel bearing of Koizumi Bridge and Shin-Aikawa Bridge, and the rubber bearings of Kesen Bridge are shown in Photos 1 to 3.

Photo 1 Damage condition of bearing support in Koizumi Bridge (tsunami-affected side (G4)). P3 pier is broken at the pier
Bridge is shown in Photo 2. The bearing support at the A1 abutment is broken at the side block bolt as seen in Photo 2(a). Considering the vertical uplift force action, the breaking part is found to agree with the weakest part in the vertical direction as shown in Fig.10(c). On the other hand, the A2 abutment bearings were broken at the set bolt (at G1 girder bearings) and the anchor bar (at G2 girder bearings); thus, the damage state was not in agreement with the weakest part calculated by our proposed method as shown in Fig.10(c).

Photo 3 shows the state of anchor bolts at the top of the Kesen Bridge substructure after the superstructure was washed away. From the behavior mechanism analysis for the Kesen Bridge superstructure, we are confirming that the lifting behavior of the superstructure on the side directly hit by the tsunami. Therefore, considering the vertical uplift force on the tsunami -side bearing supports, it can be concluded that the P1 pier bearing set bolt was broken. The part under the rubber survived as shown in Photo 3(b), and the broken part did agree with the weakest part calculated by the proposed method as evident in Fig. 10(c). Also, the A1, A2 abutment bearings and P3 pier bearings were broken at the rubber as shown in Photos 3(a), 3(b), and 3(c); thus, these damaged parts agree with the weakest part calculated by our proposed method. The P2 pier bearings are broken at the anchor bar and again, the broken part agrees with the calculated weakest part. Incidentally, the weakest part on P1 pier calculated by the methods proposed herein, was the anchor bolt, but the actual broken part was the set bolt as seen in Photo 3(b). Thus, the broken part does not agree with the weakest part.

4. TSUNAMI PROPERTY AND TSUNAMI-INDUCED BEHAVIOR OF SUPERSTRUCTURE

(1) Property of tsunami–bridge interaction

Tsunami properties are considered to vary according to the bathymetry and the geometry conditions. However, the tsunami video in the 2011 Great East Japan Earthquake supports the classification of the event as a bore tsunami and the rising-water-level tsunami as shown in Fig.11.

The bore tsunami is defined as a case where the leading bore front acts directly upon the superstructure (called acting-on-superstructure case) and if the leading bore front passes under the clearance, this is called passing case. The passing case, in the case of the rising-water-level tsunami, the tsunami acts upon the superstructure. After the tsunami impact, the tsunami overtops the superstructure. In this paper,
this state is called steady-flow. As for the rising-water-level tsunami case, the superstructure is completely submerged by this flow.

Incidentally, the bore tsunami is subclassified into undular bore, breaking bore, plunging breakers bore, etc. which may be different from the tsunami-induced force; however, but this paper uses the term “bore tsunami” for such cases. In order to evaluate the superstructural behavior during a tsunami, we need to consider the tsunami-acting mode on the superstructure. However, if there is sufficient clearance under the superstructure, turbulence flows such as a rotating vortex are generated between adjacent girders. When a turbulence flow is generated, this makes evaluation of the tsunami-induced forces more difficult because the hydrodynamic pressure between girders varies greatly. Therefore, this paper focused on the state at which the still water depth reached the bottom of the girders. It was noted that this state did not generate turbulence between girders.

(2) Supposed tsunami action mode

The properties of tsunami are classified here as bore tsunami and the rising-water-level tsunami. The bore tsunami is classified into the case acting on the superstructure and on the superstructure-submerged case.

At this time, the bore tsunami occurred immediately as the tsunami forces reached the deck height of the superstructure and the mode where the tsunami overtopped the superstructure (steady flow) generated the maximum tsunami-induced force. This paper refers to the bore tsunami reaching the deck as “Mode-I”, and where the tsunami overtops the superstructure as “Mode-II.” On the other hand, in the case of rising water-level-tsunami, the superstructure is submerged, and the maximum tsunami-induced force is generated on the completely submerged superstructure. It this case, it is referred to as Mode-III.

Where the bearing supports were broken in Mode-I, the superstructure could be washed away and overturned because the side girder impacted by the tsunami force demonstrated uplift behavior. Where the bearing supports were broken in Mode-III, we considered that the superstructure entered float behavior and washed away without overturning because all bearings were broken by the buoyancy. Mode-II is considered both washed-away type.

When evaluating the tsunami-induced force on the superstructure, the greatest tsunami-induced force for the three modes was calculated. The level of impact on the tsunami-affected bridge can be evaluated by the reaction force of the tsunami at this state and the strength of the bearing.

5. EVALUATION OF TSUNAMI-INDUCED FORCE TO BEARING SUPPORTS

(1) Tsunami-induced force on superstructure in three modes

a) Mode-I: Bore tsunami reaching deck level

The Mode-I tsunami-induced horizontal force $f_{in}$ acted directly upon the side of the girder affected by tsunami as shown in Fig.12(a). The horizontal forces include dynamic force and static force from hydrostatic pressure. Considering the force of the air entrapped between the girders, the tsunami does not act at other than the tsunami-side girders; thus, the tsu-
nami-induced force is not generated at these girders. The hydrodynamic force at the side of slab should be considered; however, the hydrodynamic pressure at the peak horizontal reaction force is small. Therefore, it was decided that the hydrodynamic pressure at the side of slab need not be considered in calculating horizontal tsunami-induced force.

In the vertical tsunami-induced force, the dynamic force \( f_{Vd} \) generated at the tsunami collision and the static force \( f_{Vs} \) from the buoyancy in the submerged part (shown in the red area) acts upwardly on the superstructure (Note: this assumes that the air between the girders remains). The flow between girders is considered to get more turbulent with the separation at the lower flange at the tsunami collision, but the authors confirmed with flume tests\(^{23}\) that the turbulent flow between girders seen in Mode-I was not generated. Therefore, within the scope of this research, it was thought that an assumption that the air was retained between girders was reasonable.

**b) Mode-II: Tsunami overtopping the completely submerged deck**

In the case of the completely submerged superstructure overtopped by the tsunami, the impact force of the tsunami hit does not act upon the superstructure, the overtopping flow (steady flow) acts on the superstructure as shown in Fig.12(b). The horizontal tsunami-induced force acts merely as the dynamic force \( f_{H} \) at the tsunami-side of girder because an equal static pressure is generated at both ends of the girders.

As for the vertical tsunami-induced force, the dynamic force \( f_{Vd} \) acted on the bottom of the overhang and the buoyancy (static force) \( f_{Vs} \) generated by the submerged superstructure acted upon the superstructure.

In this mode, the air entrapped between girders varies because of the inflow between girders by the separated flow from the lower flange, thus varying the buoyancy (static force). However, the buoyancy affects the entire superstructure including the entrapped air in the superstructure. (Note: where the velocity is small and the flow is not turbulent).

**c) Mode-III: Completely submerged superstructure**

In the case of the completely submerged superstructure shown in Fig.12(c), the horizontal tsunami-induced force does not impact \( f_{H} = 0 \) because no hydrodynamic force is generated and the hydrostatic force is canceled by the equally static pressure at both ends of the girders. Therefore, the vertical tsunami-induced force has only buoyancy where the superstructure is submerged. Incidentally, the effect of the vertical velocity might deserve consideration but flume tests have confirmed that vertical velocity has no influence on the superstructure\(^{25}\). Therefore, it was assumed that the effect of the vertical velocity need not be considered.

**(2) Evaluation of tsunami-induced force on superstructure**

Considering the tsunami-action mode as shown in Fig.12, the tsunami-induced force per unit length on the superstructure is expressed by Eq.(17)–(22). The actual phenomenon is very complicated, but in this study it was assumed that any force other than those forces shown in Chapter 5.1 could be neglected because those forces were very small.

**a) Horizontal tsunami-induced force per unit length**

\[
\begin{align*}
\text{Mode-I:} & \quad f_{H} = 0.5 \rho U_{I}^{2} h C_{M-H} C_{in} \quad (17) \\
\text{Mode-II:} & \quad f_{H} = 0.5 \rho U_{II}^{2} D C_{M-H} C_{in} \quad (18) \\
\text{Mode-III:} & \quad f_{H} \approx 0 \quad (19)
\end{align*}
\]

where, \( \rho \) is the density of water (1,000kg/m\(^3\)), \( h \) is the tsunami height (m) \((0 \leq h \leq d\), \( d \) is the main girder height (m)), \( D \) is the superstructure height, \( U_{I} \) and \( U_{II} \) are the tsunami speed (m/s). \( C_{M-H} \) and \( C_{in} \) are the modification factors of the tsunami property, and are
calculated by the hydrodynamic pressure distribution at each member. $C_{II}$ and $C_{III}$ are the drag coefficients, and the drag coefficient in Mode-I is contained in the dynamic force and the static force.

### b) Vertical tsunami-induced force per unit length

#### Mode-I:

\[
\text{Vertical tsunami-induced force per unit length} = f_{Vd} = f_{Vd1} + f_{Vd2} = 0.5 \rho U_1^2 C_{Vd1} b' C_{Vd1} + \rho gh b C_{Vd1} \tag{20}
\]

#### Mode-II:

\[
\text{Vertical tsunami-induced force per unit length} = f_{Vd} = f_{Vd1} + f_{Vd2} = 0.5 \rho U_2^2 C_{Vd2} b' C_{Vd2} + \rho g A C_{Vd2} \tag{21}
\]

#### Mode-III:

\[
\text{Vertical tsunami-induced force per unit length} = f_{Vd} = f_{Vd1} + f_{Vd2} = \rho g A \tag{22}
\]

In Mode-I and Mode-II, the first term on the right-side of Eqs.\((20)\) and \((21)\) is the dynamic force acting on the bottom of the overhang, while the second term is the buoyancy (hydrostatic force) in the submerged area. Mode-III has no hydrodynamic force acting on the bottom of the overhang because the tsunami speed is nearly zero; thus, the vertical tsunami-induced force in Mode-III has only buoyancy (hydrostatic force).

For Eqs.\((20)\), \((21)\), and \((22)\), \(g\) is the gravitational acceleration \((9.8 \text{ m/s}^2)\), \(b\) is the length between the girder ends (m), \(A\) is the superstructure-section area including the entrapped air \((\text{m}^2)\), \(h\) is the tsunami height \((0 \leq h \leq d)\), \(d\) is the main girder height (m), \(C_{Vd1}\) and \(C_{Vd2}\) are the uplift coefficients at tsunami-reached and tsunami-pasused areas. \(C_{Vd1}, C_{Vd2}, C_{Vd1}, C_{Vd2}\) and \(C_{Vd2}\) are the modification factors of the tsunami property.

#### (3) Calculation method for modification factor

##### a) Calculation equation of modification factor

The modification factors \((C_{Vd1}, C_{Vd2}, C_{Vd1}, C_{Vd2}, C_{Vd2})\) included the evaluation equation for the tsunami-induced force per unit length, as calculated by the ratio of the hydrodynamic pressure distribution at the superstructure to the assumed-equality pressure distribution. This is expressed in Eq.\((23)\):

\[
C_{ij} = \frac{P_{bi}}{P_{ai}} \tag{23}
\]

where, \(C_{ij}\) is the modification factor, the suffix \(i\) is the horizontal direction or the vertical direction, and the suffix \(j\) indicates the mode. \(P_{ai}\) is the converted value which converted equality pressure distribution into a concentrated load, and \(P_{bi}\) is the converted value for the hydrodynamic pressure at the superstructure into concentrated load.

The flow in Mode-II is made complex by the structural shape, thus the calculation of the modification factor requires careful study. Therefore, in this study, the modification factor was calculated in Mode-I through numerical analysis.

#### b) Outline of numerical analysis

In order to calculate the modification factor, the hydrodynamic pressure at the superstructure was considered in a numerical analysis using the analytical software, CADMAS-SURF/3D\(^{35}\). The authors confirmed that this software provided consistency in the reproduction analyses of flume tests\(^{36}\).

The two-dimensional analytical model is shown in Fig.13. The vertical grid size was set to 20mm (from the wave generation boundary to 1m), 5mm (0.5m before and after of the model) and 10mm (other than these areas). The horizontal grid size was set to 10mm (from the bed to 0.1m and the top to 0.2m) and 5mm (other than these areas). This study modeled some box girder bridges as shown in Fig.14 because calculations in this software for the girder bridge models were unable to provide the hydrodynamic pressure distribution at the bottom of girders. The superstructure model length needed for the calculation of the modification factor was modeled based on 4-main girders model size used in the flume test\(^{24}\) as shown in Fig.14.

This analysis showed the bore tsunami (the tsunami height increased gradually, and constant tsunami height after reaching the targeted tsunami height and tsunami velocity) based on the previous numerical analyses\(^{36}\).

The tsunami velocity was calculated by

\[
U = \frac{\zeta}{h + \zeta} \sqrt{\frac{g(h + \zeta)(2h + \zeta)}{2(h + \zeta - \eta \zeta)}} \tag{24}
\]

where, \(h\) is the still water depth (m), \(\zeta\) is the target tsunami height (m), \(\eta\) is the resistant coefficient (in this study set 1.03\(^{37}\)), and \(g\) is the gravitational acceleration \((9.8 \text{ m/s}^2)\).

In order to reproduce the tsunami (the water level rise was raised to the underside of the girders and a bore tsunami was generated), the condition of wave generation was set to 0.2m of still water depth (0.4m in actual conversion), 0.09m of target height (0.18m in actual conversion), and 2s of target time (the time to reach the target tsunami height). This analysis considered the effects of air pressure, and the vortex flow was set to the \(h-c\) 2-D equation model\(^{35}\). This analysis yielded the hydrodynamic pressure distribution at the parts (the main girder, the bottom of overhang and the girder) required to calculate the modification factor.

##### c) Determination of modification factor

Figure 15 shows the results of the hydrodynamic pressure distribution on each part. The hydrodynamic pressure distribution in Fig.15 shows the hydrodynamic pressure distribution at the base points (pressure acting on the tsunami-affected side girder: the
bottom of the girder, for the bottom of the overhang: the girder side, for the bottom of the girders: tsunami-acting-side girder bottom) at peak values.

Figure 15(a) shows the hydrodynamic pressure distribution at the girders on the tsunami acting side. The vertical axis is the distance from the bottom of girder and the horizontal axis is the ratio of the hydrodynamic pressure at each girder point to the hydrodynamic pressure at the bottom of the girder (base pressure). The hydrodynamic pressure at the girder decreases toward the top of the girders, and the hydrodynamic pressure at the top of girder is about 70% to 90% of the pressure at the girder bottom. Therefore, the hydrodynamic pressure distribution assumes that the hydrodynamic pressure decreases from the girder bottoms to the girder tops as shown in Fig.15(a), and we converted the hydrodynamic pressure distribution into a concentrated load using Eq.(23). Also, the modification factor $C_{in}$ in $b/d$ between 40/3.5 and 40/9 was calculated with the decrease rate set linearly at the tops of the girders.

$$
C_{in} = \begin{cases} 
0.95 & (b / d \leq 40/3.5) \\
0.75 + \frac{7}{400} \left(\frac{b}{d}\right) & (40/3.5 < b / d < 40/7) \\
0.85 & (40/7 \leq b / d) 
\end{cases}
$$

(25)

The hydrodynamic pressure distribution at the bottom of the overhang is shown in Fig.15(b). The horizontal axis is the distance from the girder-side, and the vertical axis is the ratio of the hydrodynamic pressure at the bottom of the overhang to the hydrodynamic pressure at the girder-side (base pressure). Although there are uneven impacts in the hydrodynamic pressure toward the tip of the overhang, the hydrodynamic pressure under $b/d=5.0/7$ is about 90%, and in $b/d=5.0/7$ is about 80% compared to the hydrodynamic pressure at the girder-side. Thus, the results supported that the hydrodynamic pressure from the main girder side to 3/4 distance is equally distributed. Other than that, there was a linear decrease rate. As a result, the modification factor is 0.987 in $b/d$ less than 5.0/7 and 0.975 in $b/d=5.0/7$; thus, the modification factor $C_{vd}$ was regarded as 1.0.

$$
C_{vd} = 1.0
$$

(26)

The hydrodynamic pressure at the bottom of the girder is shown in Fig.15(c). The horizontal axis represents the distance from the tsunami impact side, while the vertical axis represents the ratio of the hydrodynamic pressure to the hydrodynamic at the bottom of the girder (base pressure). The hydrodynamic pressure decreases away from the tsunami-acted side, while the hydrodynamic pressure opposite the tsunami-acted-side showed to be about 20%. From the results, it is assumed that the hydro-
dynamic pressure distribution linearly decreases away from the tsunami-acted side, the modification factor $C_{\text{mod}}$ is calculated by Eq.(23).

$$C_{\text{mod}} = 0.6$$  (27)

(4) Evaluation of reaction force on bearing supports

Our proposed method for evaluating the reaction force on the bearing supports due to the tsunami is based on the tsunami-induced force. In Mode I, the large reaction force acts on the tsunami acting side bearing supports. In this case, the horizontal reaction force at the tsunami-acting side bearing support, $R_{\text{in}}$, is shown in Eq.(28) and it was considered that all bearing supports took on tsunami-induced force.

$$R_{\text{in}} = \frac{f_{\text{id}}l}{N}$$  (28)

where, $l$ is the length of the superstructure, and $N$ is the number of bearing supports.

The calculations for the vertical reaction force at the bearing support considered that the hydrodynamic force ($f_{\text{hd}}$, $f_{\text{vd}}$) and the hydrostatic force ($f_{\text{hs}}$) act on the points shown in Fig.16. Assuming the superstructure to be a rigid body, the vertical reaction force $R_{\text{v1}}$ at the bearing support due to the tsunami may be estimated by Eq.(29):

$$R_{\text{v1}} = \frac{f_{\text{id}}l}{N}$$

$$+ \sum_{x=1}^{N} \frac{f_{\text{v1}}l}{N} x_i + f_{\text{id}}l$$  (29)
where, $f_{H}$ is the horizontal tsunami-induced force per unit length (kN/m), $f_{Vd}$ is the hydrodynamic force component in the vertical tsunami-induced force per unit length (kN/m), $f_{Vs}$ is the hydrostatic force component in the vertical tsunami-induced force per unit length (kN/m), $l$ is the length of the superstructure (m), $L_1, L_2, L_3$ are the distances between the center of the superstructure and action point (m), $x_i$ is the distance between the center of the superstructure and the bearing support position (m) (note: $i$ is the bearing number noted in this paper as 1, 2, ..., $n$ from the tsunami-acting-side), and $N$ is the number of bearing supports. In Mode-III, the calculations for vertical reaction force $RV_{III}$ shown in Eq.(30) considered that all bearing supports acted equally against the vertical tsunami-induced force per unit length $f_{VIII}$.

$$R_{III} = \frac{f_{H}l}{N} \quad (30)$$

This study calculated the horizontal and vertical reaction forces in Modes-I and -III. The vertical reaction force employed the larger value of Mode-I or Mode-III.

(5) Calculation of reaction force and comparison with flume test

a) Superstructure model

The reaction force at the bearing supports due to the tsunami is calculated based on the above results, and compared with the results of the flume tests with a 4-main girder model in Fig.17 (The engineer volume was converted to full scale).

The Mode-III flume tests confirmed that the vertical tsunami-induced force is nearly equal to the buoyancy. From the results of the flume test where the still-water depth reached the bottom of the girders and the bore tsunami acted upon the superstructure, the reaction force in Mode I yielded the maximum value. Therefore, the reaction force per unit length at the bearing support in Mode I was calculated.

This study calculated the reaction force per unit length at the bearing support against the tsunami beyond the bridge face where, the modification factor was set $C_{H}=0.85$, $C_{Vd}=1.0$ and $C_{Vs}=0.6$. The drag and lift coefficients were set based on the relation between the hydrodynamic pressure (at the girder and the bottom of the overhang) and tsunami speed as measured by the flume test. The results showed the calculation value was safely on the side of 95% reliability as shown in Fig.18. The drag and lift coefficients were set to $C_{D}=2.0$ and $C_{L}=1.5$.

b) Horizontal reaction force per unit length

The horizontal reaction force per unit length at the bearing support is shown in Fig.19. The horizontal axis is the tsunami speed and the vertical axis is the horizontal reaction force per unit length. The evaluation value of the horizontal reaction force calculated by Eq.(17) and Eq.(28) confirmed that it was not evaluated on the danger side vis-a-vis the flume test value.

c) Vertical reaction force per unit length

Figure 20 shows the vertical reaction force per unit length at the bearing support. The horizontal axis is the tsunami speed and the vertical axis is the vertical reaction force per unit length. The evaluation
value of the vertical reaction force calculated by Eq.(20) and Eq.(29) was confirmed to be well within safety margins according to the flume test values.

6. VERIFICATION OF APPLICABILITY BASED ON BRIDGES AFFECTED BY THE TSUNAMI OF THE GREAT EAST JAPAN EARTHQUAKE

(1) Target bridges

This study focused on tsunami-affected bridges for which superstructure and the bearing support conditions could be clearly recognized; the tsunami speed at the target bridge was developed by the video11),31)-33). As a result, nine bridges conforming to these conditions were within the authors’ scope of study. The cross-sections of these nine bridges are shown in Fig.21. The cross-section dimensions used were the representative dimensions (rounding off to tens), and those bridges were assumed to have constant sections. The dead load reaction forces shown in the drawing were used in the calculation. If no dead load reaction force is shown, it was inferred from the reaction force affecting all bearing supports.

(2) Evaluating reaction force in target bridges

The reaction force per unit length at the bearing support in target bridges is calculated by Eqs.(17) ~ (22) and Eq.(28) ~ (30). The modification factor, the drag coefficient and the lift coefficient used to calculate the reaction force were set to $C_{Dr}=0.85$, $C_{Vd}=1.00$, $C_{Vs}=0.6$, $C_{Dl}=2.0$ and $C_{Ll}=1.5$. The length of the bridge / is used as the bridge length in the case of intermediate support of continuous girder or half length in the case of the girder-end support of continuous girder and single girder. In this study, the vertical reaction force used the larger value obtained from Modes-I and -III. The horizontal reaction force in Mode-III had no value (roughly 0); thus, this study used only the horizontal reaction force in Mode-I.

The calculation results of the reaction forces for the target bridges in this study are shown in Table 2. The tsunami speed, the evaluation-targeted part (the lowest strength part), the length of the bridge and the dead load reaction force are shown in Table 2. The vertical reaction force in the bearing support in Mode-I is bigger than in Mode-III until a tsunami speed of 3.0 m/s.

(3) Evaluation of degree of influence in target bridges

Figure 22 shows the results of the comparison between the reaction force in bearing supports shown in Table 2 and the strength of the bearing supports as calculated in Chapter 3. This study compared the tsunami-affected-side bearings with the most degree of influence on the superstructure. The strength evaluation method for line bearings is not shown in this paper, therefore the Katagishi Bridge with line bearings was evaluated with the degree of influence using the maximum horizontal and vertical load based on line bearing loading tests19).

The horizontal axis in Fig.22 represents the value obtained by dividing the horizontal strength at the bearing support by the horizontal reaction force, and the vertical axis is the value obtained by dividing the vertical strength at the bearing support plus the dead load reaction force by the vertical reaction force. This paper calls these values, the tsunami horizontal resistance degree and the tsunami vertical resistance degree. When both tsunami resistance degrees exceed 1.0, the evaluation is that the bearing support is not broken and the superstructure survives. On the other hand, if either resistance degree is less than 1.0, the evaluation is that the bearing support is broken and superstructure can be washed away. If the tsunami horizontal resistance degree is less than 1.0, the
conclusion is failure from horizontal force. If the tsunami vertical resistance is less than 1.0, the evaluation is failure due to the vertical force.

Bridges annotated in blue are the survived bridge, and bridges in red are the washed away bridges due to failure of bearing supports. The reaction force at the bearing supports varies with tsunami speed and the accuracy of the evaluation equation. However, this study targeted at bridges for which the tsunami speed was derived based on video evidence\(^{(11)}\),\(^{(31)-33)}\), and the reaction force in the bearing support calculated by Eq.(28) and Eq.(30) agreed well with the results of flume tests as shown in Figs.19 and 20. Therefore, the force dispersion for the reaction force in the bearing support was not considered. On the other hand, dispersion is present in the evaluation method for the strength of bearing supports, thus this study was used in the two evaluation methods shown in Chapter 3.(4), and the range of the influence degree in target bridges in Fig.22 can be seen in the broken line. The degree of influence in Katagishi Bridge’s line bearings is shown by the points because the line

| bridge name     | evaluation part | support condition | target member | length of bridge
|-----------------|-----------------|------------------|--------------|-----------------|
| Yanoura Bridge  | A1, A2          | end support      | anchor bolt   | 36.0           |
|                 | P2,G10          | intermediate     | side block    | 308.0          |
|                 |                 |                  |              | 4.3            |
| Kesen Bridge    | A1, A2          | end support      | rubber        | 36.0           |
|                 | P4              | intermediate     | anchor bolt*1| 450.0          |
|                 |                 |                  | rubber*2      | 7.0            |
|                 |                 |                  | set bolt      |                  |
| Kosen Bridge    | A1, A2, A3      | end support      | rubber        | 67.2           |
|                 | P4              | intermediate     | anchor bolt*1| 862.0          |
|                 |                 |                  | rubber*2      | 2.0            |
| Utatsu Bridge   | A1, P1, P2      | end support      | anchor bolt   | 40.7           |
| (1th and 2th span) |              |                  | side block    | 960.0          |
|                 | P2,P7           | end support      | anchor bar    | 14.4           |
| (3th ~ 7th span) |                  |                  | (none)        | 63.6           |
|                 | P11,A2          | end support      | lower shoe    | 29.9           |
| (8th ~ 12th span) |                |                  | anchor bolt   | 437.6          |
|                 | P7,P10          | end support      | (none)        | 434.3          |
| Numata Bridge   | A1, A2, P1, P2  | end support      | anchor bar    | 23.0           |

*1: rubber ultimate tensile stress is 5.0\(\text{N/mm}^2\)  
*2: rubber ultimate tensile stress is 3.0\(\text{N/mm}^2\)  
(Advantage state)
bearing strength was evaluated based on the line bearing loading test\textsuperscript{19} – the dispersion included in the strength-evaluation equation was not considered. The degrees of influence in the Numata Bridge and the Utatsu Bridge (3th ~ 7th span) with the anchor bar are shown by the constant lines representing the tsunami vertical resistance degrees because the vertical resistance in the anchor bar is generated only by the dead load reaction force.

As shown in this figure, we were able to accurately evaluate both surviving and washed-away bridges between the non-failure zone (in blue) and the failure zone (in red and yellow) using the proposed methods of evaluation.

Here, attention is focused on the relative relationships of both tsunami resistance degrees. In the vertical resistance degree, the tsunami resistance degree on surviving bridges was greater than it was on washed-away bridges, thus the effect on the bridges is considered to be appropriately evaluated. The vertical resistance degree for Yanoura Bridge (surviving bridge) was greater than in the washed-away bridges. This is thought due to the bridge having ten main girders; thus, the reaction force per bearing was minimized as shown in Fig.21. Conversely, in the case of the low vertical resistance degree on the washed-away bridge (e.g., Shin-Aikawa Bridge), these bridges allowed easy entrapment of air from the structures as box girder bridges as well as the number of bearing supports being low. Therefore, these washed-away bridges showed lower tsunami vertical resistance degrees.

In the horizontal direction, there was no characteristic difference in the evaluation of the horizontal resistance degree between surviving bridges and washed-away bridges. However, the horizontal resistance degrees of Koizumi Bridge and Kesen Bridge were significantly lower than those of other bridges. One cause or factor is that the tsunami speed hitting these bridges was 7.0m/s higher, or nearly twice the speed hitting other bridges as shown in Table 2. The horizontal reaction force became larger and the tsunami-resistance degree was evaluated as low. From this trend, it is concluded that the degree of tsunami resistance is greatly affected by tsunami speed.

If we can properly estimate the tsunami speed at the impact, and if the structural condition of the bearing supports is clear, the proposed method allows reliable evaluation of the degree to which the bridge will be affected.

7. CONCLUSION

In order to formulate a proper evaluation method for the effects of tsunami upon bridge systems, we proposed to first study the damage mode of bearing supports and establish an evaluation method for the strength of bearing supports subjected to tsunami forces. Next, we defined how tsunami acts on the superstructure, and proposed an evaluation method.
for the reaction force of bearing supports subjected to the tsunami-induced force. Finally, the degree of effect on bridges due to tsunami was evaluated using the suggested method, wherein the usefulness of the suggested method was verified. The primary results from this study were as follows:

1) The damage mode of bearing supports was analyzed from the damage conditions of bearing supports due to the tsunami. In the results of the analysis, impacts on steel bearings considered that the failures of the set bolt, side block bolt, and pinch plate were due to vertical uplift force. For rubber bearings, it was concluded that failures of set bolts were caused by the vertical uplift force. However, the rubber failures may be due to either forces because there was significant variance in the tensile strengths of rubber.

2) The calculation result of the transverse strength in the bearing support using the suggested method was confirmed as valid, and the main weakest part was the anchor bolt. This finding regarding the weakest part agreed with the damage mode of the bearings in Koizumi Bridge (P1, P5) as shown in Photo 1 and Kesen Bridge (P2) as shown in Photo 3.

3) In the calculation result of the vertical strength calculation in the bearing support, it was confirmed that the main weakest part for steel bearings was the side block bolt, and this finding agreed with the damage mode of the Koizumi Bridge (A1, A2 abutment, and P1, P5 pier) and the Shin-Aikawa Bridge (A1 abutment) as shown in Photos 1 and 2. For the case of rubber bearings, the weakest part differed according to the tensile strength of the rubber. In Kesen Bridge, the weakest part at A1, A2 abutment, P3 pier and P1, P2 piers is the anchor bolt or the rubber, while the weakest part at P4 pier is the set bolt or the rubber. These results generally agreed with the damage mode as shown in Photo 3.

4) This study suggested the evaluation equation of the reaction force in the bearing support due to the tsunami as Eq.(28) ~ (30), then compared with the measured values from the flume test. The result of the comparison confirmed that the value calculated by the modification factor shown in Eq.(25) ~ (27) (using $C_{Df}=2.0$ and $C_{I}=1.5$) was not within the danger zone.

5) In the result of evaluation of the degree of influence in bridges affected by the tsunami using the suggested evaluation method, the method reasonably segregated surviving bridges from the washed-away bridges.

6) Regarding the relationship of the vertical resistance degree, the ratio for surviving bridges was bigger than for washed-away bridges, thus the tsunami-resistance degree was appropriately evaluated. On the other hand, the horizontal tsunami-resistance degree demonstrated no great difference between surviving and washed-away bridges. However, the tsunami-resistance degree was lower for Koizumi Bridge and Kesen Bridge than for other bridges, because the tsunami speed around Koizumi Bridge and Kesen Bridge was about twice higher than the speed around the other bridges and the horizontal reaction force reached large values.

7) If the tsunami speed at tsunami impact is known and the structure of bearing supports becomes clear, the proposed methods can reasonably evaluate the tsunami-resistance degree in existing bridges.

REFERENCES

1) Reconnaissance Report on Damage to Road Bridges by the 2011 Great East Japan Earthquake, Technical Note of National Institute for Land and Infrastructure Management, No. 814, Technical Note of Public Works Research Institute, No. 4295, 2014.

2) Arikawa, T., Watanabe, M. and Kubota, K. : Stability of bridge girder under tsunami, Journal of Japan Society of Civil Engineers, Ser. B2 (Coastal Engineering), Vol. 69, No. 2, pp. 1_91-1_915, 2013.

3) Shimizu, H. and Shoji, G. : Relationship between tsunami fluid force and tsunami velocity acting on a bridge deck, stability of bridge girder under tsunami, Journal of Japan Society of Civil Engineers, Ser. B2 (Coastal Engineering), Vol. 69, No. 2, pp. 1_941-1_945, 2013.

4) Yamauchi, K., Ueshima, S. and Kosa, K. : Study on evaluation of tsunami wave force to bridge and reduced wave method, Proceedings of Japan Society of Civil Engineers 2012 Annual Meeting, I-475, pp. 949-950, 2012.

5) Hamai, S., Kosa, K., Sasaki, T. and Sato, T. : Study on vertical wave force acting on bridge due to solitary wave tsunami, Proceedings of JCI Annual Convention, Vol. 36, No. 2, pp. 565-570, 2014.

6) Kawasaki, Y., Izuno, K., Kushima, N., Yamanaka, T. and Yotsui, S. : Experimental study on a shape of baffle plate for reducing hydrodynamic forces by tsunami, Journal of Japan Society of Civil Engineers, Ser. A1 (Structural Engineering & Earthquake Engineering (SE/EE)), Vol. 70, No. 1, pp. 129-136, 2014.

7) Tanabe, S., Asai, M., Nakao, H. and Izuno, K. : Evaluation of fluid force acted on a bridge girder during tsunami by using a particle method and its validation, Journal of Structural Engineering, Vol. 60A, pp. 293-302, 2014.

8) Kataoka, S., Kaneko, M., Matsuoka K., Nagaya, K. and Unjoh, S. : Earthquake-tsunami damage simulation of a highway bridge of which superstructure and a pier were washed out, Journal of Japan Society of Civil Engineers, Ser. A1 (Structural Engineering & Earthquake Engineering (SE/EE)), Vol. 69, No. 4 (Journal of Earthquake Engineering, Vol. 32), pp. I_932-I_941, 2013.

9) Shigihara, Y., Fujima, K. and Shoji, G. : Numerical simulation of tsunami force acting on the bridge, Journal of Japan Society of Civil Engineers, Ser. A1 (Structural Engineering & Earthquake Engineering (SE/EE)), Vol. 65, No.
1) Azadbakht, M. and Cheugg, K. F.: Development of a guideline for estimating tsunami forces on bridge superstructures, Journal of Japan Society of Civil Engineers, Ser. B3 (Ocean Engineering), Vol. 69, No. 2 (Annual Journal of Civil Engineering in the Ocean), Vol. 29, pp. I 359-I 364, 2013.

2) Sasaki, T., Kosa, K., Jinguiji, H. and Sato, T.: Damage analysis in Koizumi area due to tsunami triggered by the 2011 off the Pacific coast of Tohoku earthquake, Journal of Japan Society of Civil Engineers, Ser. B2 (Coastal Engineering), Vol. 69, No. 2, pp. I 821-I 825, 2013.

3) Jinguiji, H., Kosa, K., Sasaki, T. and Sato, T.: Tsunami damage evaluation of Kesen Bridge by using video and simulation analysis, Journal of Structural Engineering, Vol. 60A, pp. 271-281, 2014.

4) Nakao, H., Moriya, Y., Enomoto, T. and Hoshikuma, J.: Estimation for property of tsunami and tsunami-induced effect on bridge around Miyako Bridge, Journal of Japan Society of Civil Engineers, Ser. A1 (Structural Engineering & Earthquake Engineering (SE/EE)), Vol. 71, No. 4 (Journal of Earthquake Engineering, Vol. 34), pp. I 317-I 328, 2015.

5) Sumimura, T., Guangfeng, Z., Nakao, H. and Hoshikuma, J.: Experimental study on resistance capacity of steel bearing supports in bridges under tsunami-induced loading, Journal of Japan Society of Civil Engineers, Ser. A1 (Structural Engineering & Earthquake Engineering (SE/EE)), Vol. 69, No. 4 (Journal of Earthquake Engineering, Vol. 32), pp. I 102-I 110, 2013.

6) Hoshikuma, J.: Development and measurement on bridge behavior due to tsunami-induced force, Report on the Great East Japan Earthquake, 2013. http://www.nilim.go.jp/lab/bbg/saigai/h23tohoku/houkoku3/houkoku3.htm

7) Japan Road Association: Road Bridge Bearing Manual, 2004.

8) Japan Road Association: Standard Design for Highway Bridges Bearings (Sliding bearing), 1993.

9) Abe, M., Yoshida, J., Fujino, Y., Morishige, Y., Uno, S. and Usami, S.: Experimental investigation of ultimate behavior of metal bridge bearings under seismic loading, Journal of Japan Society of Civil Engineers, No. 773/I-69, pp. 63-78, 2004.

10) Moriya, Y., Nakao, H. and Hoshikuma, J.: Horizontal and vertical strength of line bearing supports in existing steel girder bridges, Proceedings of Conference on Earthquake Engineering, Vol. 35, Paper No. A14-825, 11 pages (CD-ROM), 2015.

11) Mander, J. B., Kim, D.-K., Chen, S. S. and Premus, G. J.: Response of Steel Bridge Bearings to Reversed Cyclic Loading, Technical Report NCEER-96-0014, 1996.

12) Shimizu, H., Kosa, K., Sasaki, T. and Takeda, S.: Tsunami damage analysis in highway bridges, Journal of Structural Engineering, Vol. 58A, pp. 366-376, 2012.

13) Yim, S. C., Boonintra, S., Nimmala, S. B., Winston, H. M., Azadbakht, M. and Cheugg, K. F.: Development of a guideline for estimating tsunami forces on bridge superstructures, Oregon Department of Transportation, 2011.

14) Nakao, H., Guangfeng, Z., Sumimura, T. and Hoshikuma, J.: Effect of behavior of superstructure due to tsunami speed, Proceedings of the 16th Symposium on Performance-Based Seismic Design Method for Bridges, pp. 421-428, 2013.

15) Guangfeng, Z., Nakao, H. and Hoshikuma, J.: Study on behavior of bridge under tsunami-induced force, Proceedings of the 15th Symposium on Performance-Based Seismic Design Method for Bridges, pp. 97-102, 2012.

16) Nakao, H., Guangfeng, Z. and Hoshikuma, J.: Study on behavior of bridges subjected to buoyancy due to water level rises gradually by tsunami, Proceedings of the 15th Symposium on Performance-Based Seismic Design Method for Bridges, pp. 151-154, 2012.

17) Experiment study on resistant character of steel bearing against tsunami, Technical Note of Public Works Research Institute, No. 4319, 2016.

18) Nishi, H. and Takahashi, Y.: Evaluation of ultimate state of bearing support, Proceedings of the 17th Symposium on Performance-Based Seismic Design Method for Bridges, pp. 333-340, 2014.

19) Yasumatsu, T., Ishida, H., Tanaka, K. and Murayama, Y.: Earthquake resistant characteristics of anchor bars of seismic stopper, Journal of Japan Society of Civil Engineers, Vol. 633/I-49, pp. 81-92, 1999.

20) Sasaki, T., Kosa, K., Jinguiji, H. and Sato, T.: Tsunami characteristic analysis of Kamaishi area by the video analysis, Journal of Japan Society of Civil Engineers, Ser. B2 (Coastal Engineering), Vol. 70, No. 2, pp. I 326-I 330, 2014.

21) Sasaki, T., Kosa, K. and Yulong, Z.: Damage analysis of bridge based on the ratio of resistance force of the girder and acting force of the tsunami, Journal of Structural Engineering, Vol. 59A, pp. 417-427, 2013.

22) Kataoka, S.: Study and view on tsunami-induced force to structure, Report on the Great East Japan Earthquake, http://www.nilim.go.jp/lab/bbg/saigai/h23tohoku/houkoku3/houkoku3.htm

23) Urgent Survey for 2011 Great East Japan Earthquake and Tsunami Disaster in Port and Coasts, Technical Note of the Port and Airport Research Institute, No. 1231, 2011.

24) Coastal Development Institute of Technology: Study and Development of CADMAS-SURF/3D, 2010.

25) CoastBuild 2011, Comite de la Brique, 2011.

26) Fukui, Y., Nakamura, M., Shirai, H. and Sasaki, Y.: Study of tsunami (I) -About bore-tsunami velocity-, Proceedings of the 9th Coastal Engineering, pp. 44-49, 1962.

(Received September 27, 2018)