TSUNAMI LOAD EVALUATION BASED ON DAMAGE OBSERVATION AFTER THE 2011 GREAT EAST JAPAN EARTHQUAKE

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ABSTRACT: To design and construct buildings resistive to tsunami loads, quantitative evaluations of tsunami load applicable to structural design is most essential. The practical design load for tsunami shelters proposed by The Building Center of Japan in 2004 were examined through surveys of structures after the Indian Ocean Tsunami in December 2004. Nonetheless researches on tsunami load against structures based on damage observations are yet insufficient. In this paper, structures that experienced the Great East Japan Earthquake were surveyed, and the relationship between their damage, strengths, and inundation depth is quantitatively investigated to examine the design load.

Key Words: The Great East Japan Earthquake, Tsunami, Tsunami shelter, Design load

INTRODUCTION

To design and construct buildings resistive to tsunami loads, quantitative evaluations of tsunami load applicable to structural design is most essential. The design guidelines for tsunami shelters were developed by a task committee under the Japanese Cabinet Office in 2005 referring "Structural Design Method of Building to Seismic Sea Wave" (Okada et al. 2004a and 2004b), which introduced a formula to compute tsunami loads expected to act on shelters constructed on coastlines (JCO 2005). The formula was developed primarily based on laboratory tests of 2-dimensional scaled model (Asakura et al. 2000) and examined through surveys of structures after the Indian Ocean Tsunami in December 2004 (Nakano 2008). However, since researches on tsunami loads against structures based on damage observations are yet insufficient and tsunami damage caused by the 2011 Great East Japan Earthquake was devastating, quantitative evaluations of tsunami loads are currently in urgent need in terms of tsunami disaster prevention in the future. The authors therefore made extensive damage surveys of structures that experienced the tsunami caused by the 2011 Great East Japan Earthquake to investigate the relationship between their lateral strength and observed damage, and to verify the appropriateness of the design formula. In this paper, the outline of the damage surveys and investigated results on design tsunami loads are presented.

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OUTLINE OF DAMAGE SURVEYS

Surveyed areas
Damage surveys were made in Tohoku area (from Hachinohe city in Aomori Prefecture to Soma city in Fukushima Prefecture as shown in **Figure 1**) from the beginning of April through the end of June, 2011.

Figure 1. Surveyed areas

Survey strategy
Surveys were made to review the overall damage in the areas, and to record structural dimension and reinforcement arrangement etc. to further investigate the relationship between their lateral strength and tsunami load that acted on them since they met the following three conditions:

(1) The lateral strength of the surveyed structures could be simply estimated based on the structural properties obtained on site, because (i)their sectional properties (cross-sectional size, reinforcement arrangement, etc.) were measured; (ii)their damage (or collapse) mechanism was simple and the boundary between damaged and intact part of the structure was not complicated; and (iii)they were small and/or regular enough in their plan and height that their lateral strength could be calculated through simple modeling and assumptions.

(2) The tsunami inundation depth was clearly found on the surveyed site through water marks left on building's walls, where it was defined and measured as the water depth above the ground level at the building's site.

(3) The tsunami load could be simply estimated because the surveyed structures were located in areas close to the coastlines and the direct effects by tsunami attack were the primary source of the damage.

In this paper, the measured tsunami inundation depth η_m corresponding to the design tsunami inundation depth h (m), which will be described later, was defined not to be affected by the local water splash-up on the front face of buildings. This is because the design tsunami inundation depth is generally simulated neglecting local effects of buildings. The measured tsunami inundation depth η_m is, therefore, defined as:

(1) the tsunami trace found on a rear or side walls of a building in flatland areas from Ishinomaki city to Soma city because it was higher on a front face than on the other faces due to water splash-up, and

(2) the highest tsunami trace around a building in saw-tooth Sanriku area from Kuji city to Onagawa town because no major differences in tsunami height were found among traces on all sides of exterior walls. This is due primarily to the fact that the flow velocity was not high enough at its maximum inundation depth in Sanriku area to cause water splash-up on the front face as evidenced by the tsunami videos recorded during the event.

Detailed information recorded on investigated structures
Photo 1 shows the typical structures investigated in this study. Considering three conditions for detailed surveys described earlier, more than 130 structures including (a)buildings with simple configuration, (b)fence walls, (c)RC or masonry columns (bridge piers, gate piers, etc.), (d)stone monuments, (e)seawalls, and (f)steel fences, were investigated and detailed structural data were collected. They include (1)locations with GPS data, (2)topographical information of the site, (3)use and type of structure (RC, CB (concrete block), stone, etc.), (4)damage pattern, (5)tsunami inundation depth η_m, (6)structure and/or member dimension (B x D x H, wall thickness, etc.), (7)reinforcement arrangement (diameter, spacing, cover concrete depth, etc.), (8)general view photos, and (9)structural configurations.

Collapsed building (a)   Survived building (a)   Collapsed fence wall (b)   Survived fence wall (b)
(Rikuzentakata, RC 1-story)   (Onagawa, RC 4-story)   (Watari, CB)   (Ofunato, RC)

Collapsed Railway bridge (c)   Collapsed gate pier (c)   Overturned monument (d)   Collapsed seawall (e)
(Tanohata, RC)   (Watari, stone)   (Watari, stone)   (Yamada, RC)
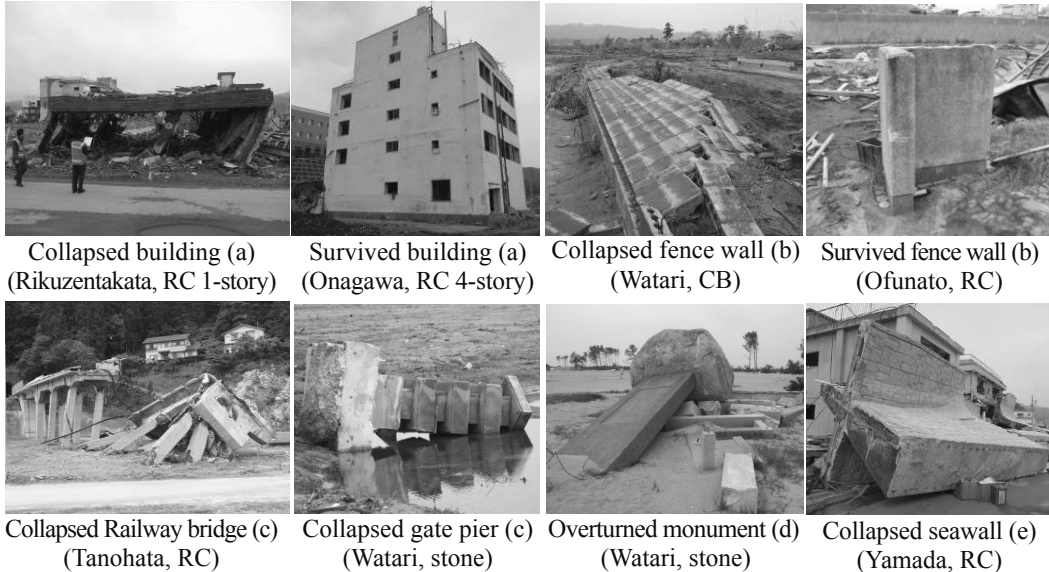
Photo 1. Typical investigated structures (Letters (a) to (e) show categories of structures)

EVALUATION OF LATERAL STRENGTH OF INVESTIGATED STRUCTURES

Lateral strength of buildings
The lateral strength of the buildings categorized in (a) as exemplified in Photo 1 is evaluated based on the first level screening of seismic evaluation procedure which is generally applied in Japan (JBDPA 2001), using the measured structure dimension and the material test results. Buoyant forces are neglected in the evaluation because overturned buildings are not discussed in this paper.

Lateral strength of simple structures
According to the damage and failure mode observed, the cracking strength M_c, the flexural yielding strength M_y, the ultimate flexural strength at rebar fracture M_u, the overturning strength M_{OT}, the sliding strength P_s are calculated for simple structures categorized in (b) through (e) described earlier, using the conventional equations widely applied in Japanese design practice and the material test results. M_{OT} is applied for the overturned structures such as gravity-type seawalls, gate piers, stone monuments, etc. and P_s is applied for the slid stone monument. The buoyant force is neglected in calculating M_{OT} and P_s herein, because it is negligibly small when a tsunami applies impulsive force to a structure.

COMPARISON BETWEEN TSUNAMI LOAD AND OBSERVED DAMAGE

Definition of tsunami pressure distribution acting on structures
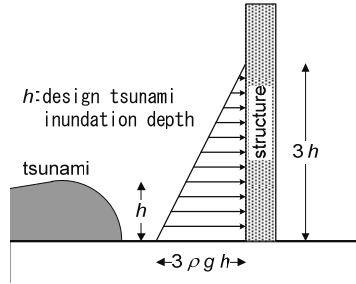
In the guidelines (JCO 2005), the design tsunami load is defined by Equation (1). In the subsequent investigations, Equation (2) that is analogous to Equation (1) is first defined, and the water depth coefficient a is evaluated (Nakano 2008) setting the lateral strength of the investigated structures equal to the tsunami load computed from Equation (2):
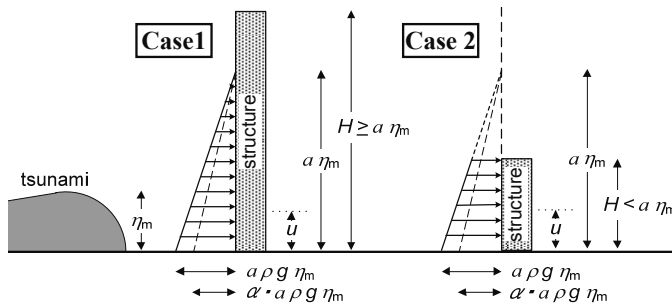
$$q_z = \rho\, g\, (\, 3\, h - z\,) \qquad\qquad (0 \leq z \leq 3h) \qquad\qquad (1)$$
$$p_z = \rho\, g\, (\, a\, \eta_m - z\,) \qquad\qquad (0 \leq z \leq a\eta_m) \qquad\qquad (2)$$

where q_z (kN/m^2) is the design tsunami pressure acting on a structure at a distance z (m) above the ground level defined in the guidelines (JCO 2005); ρ (t/m^3) is the mass per unit volume of water (1.0 assumed herein); g (m/s^2) is the gravity acceleration; h (m) is the design tsunami inundation depth; p_z (kN/m^2) is the tsunami pressure acting on a structure at a distance z (m) above the ground level defined in this study to investigate a rational value of a; η_m (m) is the measured tsunami inundation depth; and a is the water depth coefficient defined by the ratio of the water depth $a\eta_m$ equivalent to the structure's strength to the measured tsunami inundation depth η_m (i.e., $a = a\eta_m / \eta_m$).

Figure 2 illustrates the background concept employed in Equation (1). The design tsunami pressure distribution acting along the structure's height is assumed a triangular shape with the height reaching 3 times of the design tsunami inundation depth h (i.e., the pressure at the bottom is assumed 3 times of the hydrostatic pressure) based on the experimental results (Asakura et al. 2000). In calculating the coefficient a, two typical cases of inundation depth and structure's height, which can be found in the guidelines (JCO 2005), are taken into consideration as shown in Figure 3 since they are the basic patterns of tsunami attack to existing structures in the surveyed areas. The tsunami pressure is uniformly reduced by the ratio α defined as [1 - (opening area)/(loaded area)] as shown in Figure 3 when the building has openings.

Figure 2. Design tsunami pressure distribution (JCO 2005)

Figure 3. Measured tsunami inundation depth η_m, structure's height H, and tsunami pressure distribution p_z

Procedure to compute the water depth coefficient a

The coefficient a can be computed assuming that the lateral strength of an investigated structure is equal to the overall tsunami load acting on it under the pressure distribution along its height defined by Equation (2) and **Figure 3**. The coefficient a therefore denotes the ratio of the equivalent water depth $a\eta_m$ corresponding to the structure's lateral strength under a triangular hydrostatic pressure profile to the measured tsunami inundation depth η_m. If the coefficient a categorizes collapsed and survived structures at its value of 3, one can say that Equation (1) with a equal to 3 is a rational design formula to compute the tsunami load. The procedure to compute the coefficient a is described in detail below (Nakano 2008).

1. Compute the lateral strength of investigated structures considering their failure mode as shown in the previous chapter.
2. Then compute shear force or bending moment acting at the failure point u (defined as the distance between the failure point and the ground surface) assuming the tsunami pressure distribution as defined in Equation (2). Setting the force or moment at the height u equal to the lateral strength obtained in step 1. above, evaluate the equivalent water depth $a\eta_m$ corresponding to the strength. Note that the tsunami pressure above structures is neglected and the depth $a\eta_m$ is evaluated assuming the trapezoidal instead of triangular pressure distribution in computing the force or moment as shown in case 2 of **Figure 3**.
3. Finally compute the coefficient a, which is defined as the ratio of equivalent water depth $a\eta_m$ to measured tsunami inundation depth η_m (i.e., $a = a\eta_m / \eta_m$).
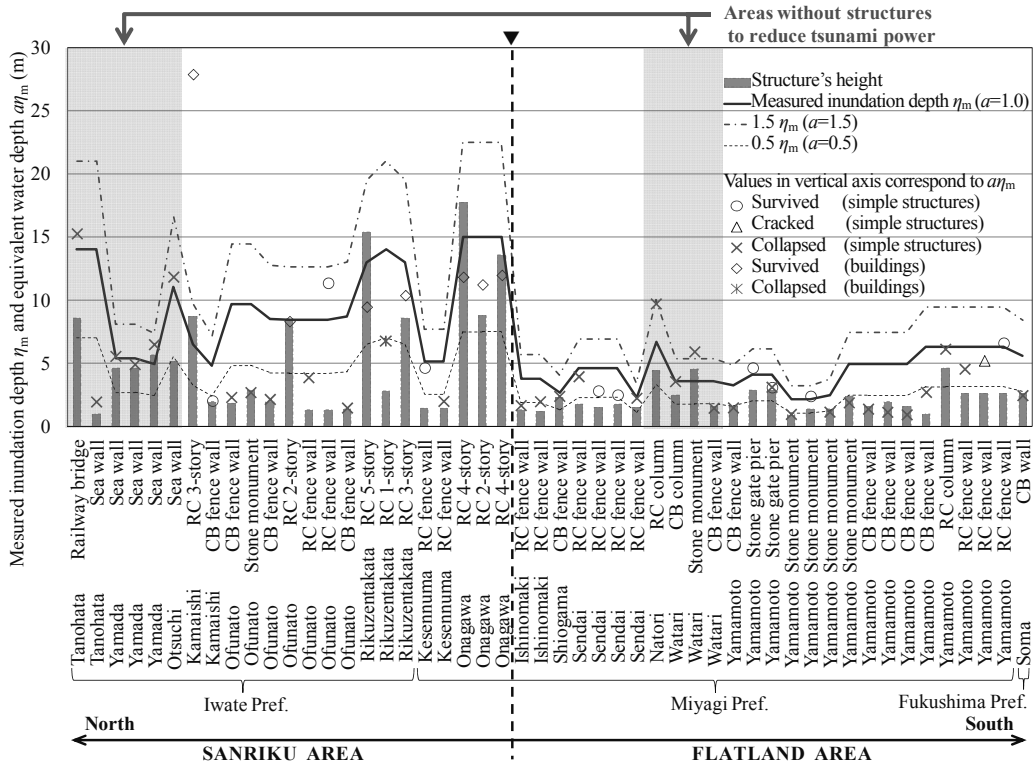
Tsunami load evaluation based on structure's damage

Of all surveyed structures, 43 simple structures and 8 RC buildings are employed to evaluate the coefficient a. To overview the relationship between damage, measured tsunami inundation depth η_m, and other data, they are plotted from north to south in **Figure 4**, starting from left, together with their structural type, height, $0.5\eta_m$, and $1.5\eta_m$. Note that $0.5\eta_m$, η_m, and $1.5\eta_m$ in the figure correspond to 0.5, 1.0, and 1.5 in coefficient a. A dashed line in **Figure 4** shows the boundary between ria coast area (Sanriku area) and flatland area such as Sendai plain. As can be found in the figure, the measured tsunami inundation depth η_m in Sanriku area is found generally higher than that in flatland area. Each plot in **Figure 4** explains that the collapsed structures (designated as x and *) were exposed to the force higher than the hydrostatic force due to the water depth $a\eta_m$ (corresponding to the structure's strength) plotted in the vertical axis, while the survived structures (designated as o and ◊) were exposed to the force lower than that, and the cracked structures (designated as Δ) were just exposed to the hydrostatic force due to $a\eta_m$. Therefore, the actual tsunami load can be found as the boundary value between collapsed (x and *) and survived (o and ◊) structures.

In general, structures with high coefficient a survived and those with low coefficient a collapsed as shown in **Figure 4**. Although one may expect higher value of a in flatland area due to higher flow velocity, no such a tendency is found and the topographical difference may not be the major parameter to find a rational value of a.
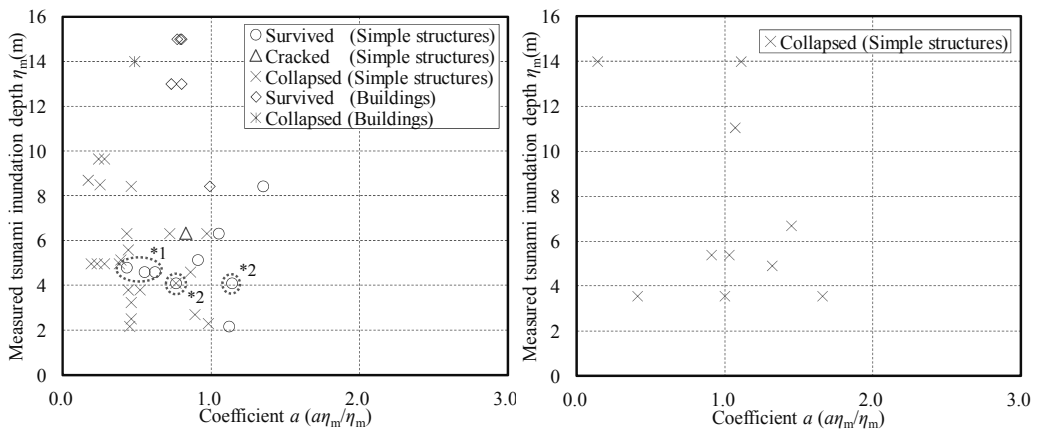
It should be noted, however, that some structures located just close to the coastlines such as seawalls collapsed even though their strengths are high as shown in the hatched area of **Figure 4**. Assuming that the tsunami load differs depending on the site environment, the surveyed structures are then divided into two groups, i.e., those in areas with or without structures on the sea side that are deemed effective to reduce the tsunami power such as breakwater. **Figure 5** shows the relationship between the coefficient a and the measured tsunami inundation depth η_m for each area with different site environment.

Figure 5(a) shows structures other than those plotted in the hatched area in **Figure 4**. They are located in Kamaishi city, Ofunato city, and Onagawa town where breakwaters are provided at its bay mouth; Rikuzentakata city, Ishinomaki city, Sendai city, etc. where breakwaters and seawalls which are considered high enough to reduce the tsunami power are provided; Kesen-numa city where the bay holds Hachigasaki area off the coast which served as a natural breakwater. Note that the data denoted by *1 are structures oriented parallel to the direction of the tsunami flow and survived

despite the coefficient *a* was much smaller than 1 due to the water pressure acting on both front and rear sides of them. Also note that the structures denoted by *2 are stone gate piers embedded in the ground but its contribution to the overturning resistance is neglected for simple calculation and therefore their actual coefficient *a* shuold be larger than the value shown in **Figure 5(a)**. Considering those mentioned above, **Figure 5(a)** shows that the coefficient *a* to discriminate between collapsed and

Figure 4. Relationship between measured inundation depth, equivalent water depth and structure's damage

(a) Areas with structures to reduce tsunami power   (b) Areas without structures to reduce tsunami power
Figure 5. Computed coefficient *a* vs. measured tsunami inundation depth η_m

survived structures is around 1 (i.e., equivalent to the measured tsunami inundation depth η_m) when the measured depth η_m is smaller than 10m. **Figure 5(a)** also shows that the buildings whose coefficient *a* is smaller than 1 survived when η_m is larger than 13m. It implies that the tsunami load was smaller than the hydrostatic force of inundation depth η_m, because the water had flown all around the buildings when the inundation depth reached η_m and loaded on all sides of the building.

   Figure 5(b) shows structures plotted in the hatched area in **Figure 4**. They are located in areas without structures to reduce the tsunami power. **Figure 5(b)** shows that the coefficient *a* to discriminate between collapsed and survived structures is above 1.7. Note that the upper bound of the coefficient *a* cannot be determined from **Figure 5(b)** because survived structures are not plotted in **Figure 5(b)**. **Table 1** summarizes the results found in the above discussions.

   It should be noted that the results based on the damage observations after the 2004 Indian Ocean Tsunami (Nakano 2008) concluded that the boundary coefficient *a* lies in the range of 2 to 2.5, which is much larger than the result obtained in this study. This is probably because structures investigated after the Indian Ocean Tsunami were located just close to the coastlines with no built environment to reduce tsunami power and high tsunami waves directly attacked the structures.

Table 1. Coefficient *a* to discriminate collapsed and survived structures

	Areas with structures to reduce tsunami power	Areas without structures to reduce tsunami power
Coefficient *a*	1*	≥ 1.7

* *a* < 1 when $\eta_m \geq 13$

VALIDATION OF THE COEFFICIENT *a* BASED ON DRAG FORCE

Drag force evaluation

In the previous discussions, the tsunami pressure distribution acting along the structure's height is assumed a triangular similar to the hydrostatic pressure as expressed by Equation (2), which is supposed to be applied for impulsive force on onshore structures. However the 2011 Great East Japan Earthquake evidenced through numerous recorded videos that structures were gradually inundated and may have been affected more significantly by drag force rather than impulsive force. In this paper, the tsunami load is evaluated based on drag force and compared to the result of the previous chapter.
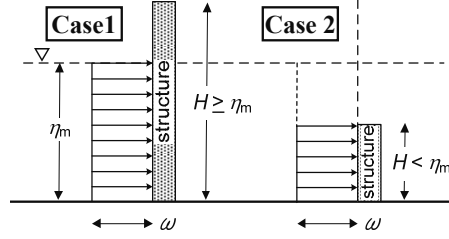
   The drag forces F_D is generally expressed by Equation (3).

$$F_D = \frac{1}{2}\rho C_D u^2 A_D \tag{3}$$

where F_D (kN) is the drag force; ρ (t/m^3) is the mass per unit volume of water (1.0 assumed herein); C_D is the drag coefficient; u (m/s) is the flow velocity; and A_D (m^2) is the loaded area of a structure in the plane normal to the direction of flow. Assuming the drag coefficient C_D 2.0 for the quadrilateral structures surveyed (JSCE 1971), Equation (3) leads to Equation (4).

$$F_D = \rho\, u^2\, A_D \tag{4}$$

   Assuming that the tsunami pressure is uniformly distributed as shown in **Figure 6**, the pressure ω_R under which the overall tsunami load is equal to the structure's strength considering their failure mechanism is obtained. Then the tsunami load *V* equal to the structure's strength is expressed by Equation (5). Note that the tsunami pressure above the structures is neglected as shown in case 2 of **Figure 6**.

Figure 6.   Measured inundation depth η_m, structure's height H, and tsunami pressure distribution ω

$$V = \omega_R A_D \tag{5}$$

Setting F_D equal to V, the flow velocity u_R equivalent to the structure's strength is expressed by Equation (6).

$$u_R = \sqrt{\omega_R / \rho} \tag{6}$$

Then the Froude number F_{rR} equivalent to the structure's strength is expressed by the equivalent flow velocity u_R and the measured tsunami inundation depth η_m (Equation (7)).

$$Fr_R = \frac{u_R}{\sqrt{g\eta_m}} = \frac{\sqrt{\omega_R / \rho}}{\sqrt{g\eta_m}} \tag{7}$$

From the discussion above, the equivalent flow velocity u_R and the Froude number F_{rR} are obtained from Equation (6) and Equation (7) using the equivalent tsunami pressure ω_R. To estimate the actual tsunami pressure, flow velocity, and Froude number, the equivalent values to discriminate between collapsed and survived structures are estimated as discussed earlier.

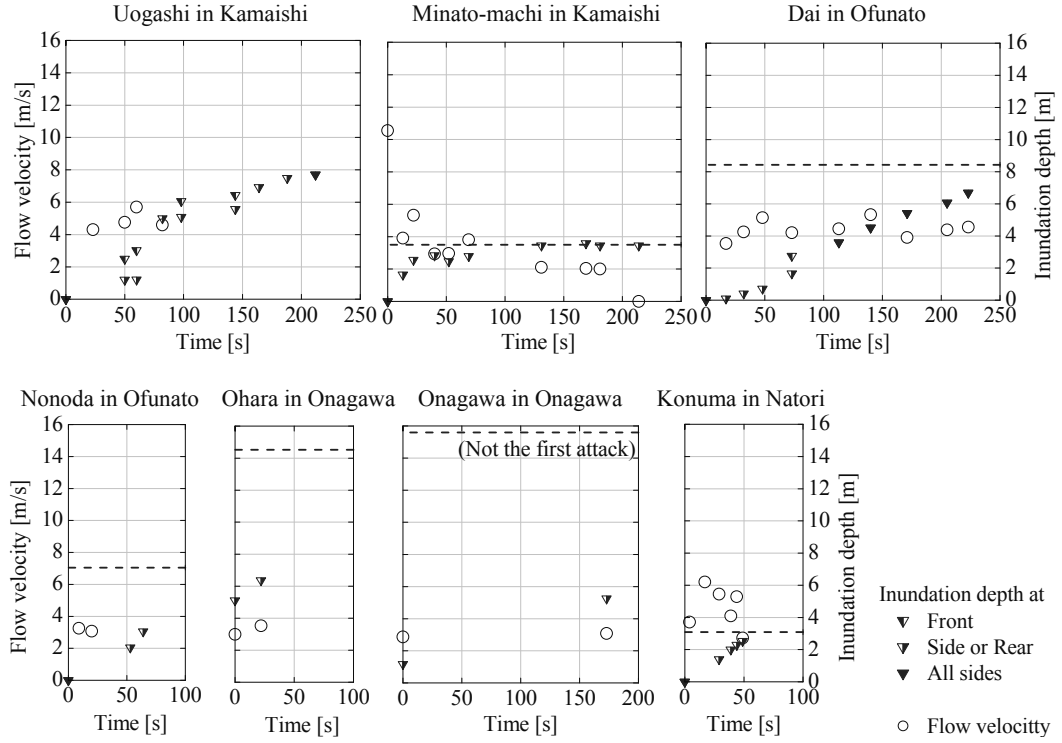***Tsunami load evaluation based on relationship between structure's damage and drag force***
Figure 7 shows the equivalent tsunami pressure ω_R, flow velocity u_R, and Froude number F_{rR} obtained from Equations (5), (6), and (7). As was done earlier in this study, the structures are divided into two groups, i.e., areas with or without structures to reduce the tsunami power. Then they are plotted from north to south in **Figure 7**, starting from left, showing their structural type, damage category, and the equivalent water depth $a\eta_m$. Alphabets in **Figure 7** show the observed failure mode of each structure: "Y", "U", "S", and "O" correspond to rebar yielding, rebar fracture, sliding, and overturning, respectively. To estimate the tsunami pressure, flow velocity, and Froude number, the boundary values to discriminate between collapsed and survived structures are found and hatched as shown in **Figure 7**. Note that the hatched horizontal lines in the area without structures to reduce the tsunami power corresponds to the lower bound because no data on survived structure in the area are available in this study.

The tsunami pressure acted on the structures, as shown in **Figure 7(a)**, is more than 85kN/m^2 where they have no structures to reduce the tsunami power, while it lies in the range of 15 to 30kN/m^2 where they have structures to reduce the tsunami power, excluding the data denoted by *1.

The flow velocity, as shown in **Figure 7(b)**, is more than 9m/s where they have no structures to reduce the tsunami power, while it lies in the range of 4 to 6m/s where they have structures to reduce the tsunami power. To validate the estimated flow velocity in **Figure 7(b)**, it is compared to the velocity of the floating objects recorded in the tsunami videos in the following 7 points: Kamaishi city (2 points), Ofunato city (2 points), Onagawa town (2 points) in Sanriku area and Natori city (1 point) in flatland area. All of them have structures to reduce tsunami power such as breakwaters or seawalls. Note that one record in Onagawa town corresponds to the second or later attack, while the other 6

records are confirmed the first attack since the first arrival of tsunami is recorded. The measured velocities are plotted in **Figure 8** only when the floating objects appear on the videos. As can be found in **Figure 8**, the flow velocity lies in the range of 3 to 6m/s regardless the inundation depth and consistent with the estimated result (4 to 6m/s) based on drag force.

Figure 7. Relationship between equivalent tsunami pressure, flow velocity, Froude number and structure's damage

Figure 8. Measured flow velocity and inundation depth derived from the tsunami recorded videos

The Froude number F_r, as shown in **Figure 7(c)**, is more than 1.27 where they have no structures to reduce tsunami power. On the other hand, it lies in the range of 0.5 to 0.65 in Sanriku area and it is around 0.8 in flatland area where they have structures to reduce tsunami power. The Froude numbers in Sanriku area is rather smaller than that in flatland area with structures to reduce tsunami power although the flow velocity in Sanriku area is the same or even higher than that in flatland area. Therefore, the flow velocity in Sanriku area is found not so high despite the high inundation depth.

Evaluation of the coefficient a based on drag force

Since the estimated flow velocity is found consistent with that derived from the tsunami recorded videos, the coefficient a is then evaluated in terms of the Froude number obtained earlier and compared to those previously calculated.

Given that the equivalent water depth $a\eta_m$ is lower than a structure with no openings, the wave force F_x under a triangular hydrostatic pressure profile by Equation (2) and the drag force F_D can be expressed as Equation (8) and Equation (9), respectively, for a unit width (1m) of a structure.

$$F_x = \frac{1}{2}(a\eta_m \times 1)a\rho g\eta_m = \frac{a^2}{2}\eta_m{}^2\rho g \tag{8}$$

$$\begin{aligned}F_D &= \rho u^2 A_D \\ &= \rho u^2(\eta_m \times 1) = F_r^2\eta_m{}^2\rho g\end{aligned} \tag{9}$$

Setting F_x equal to F_D, Equation (10) is obtained. Then the relationship between the coefficient a and Froude number F_r is expressed by Equation (11).

$$\frac{a^2}{2} = F_r^2 \tag{10}$$

$$a = \sqrt{2} \times F_r \tag{11}$$

Table 2 shows the result of the coefficient a calculated using Equation (11) with the Froude number estimated above. The coefficient a of **Table 2** in each case with and without structures to reduce tsunami power roughly corresponds to the result in **Table 1**.

Table 2. Froude number F_r and coefficient a to discriminate collapsed and survived structures estimated based on drag forces

	Areas with structures to reduce tsunami power		Areas without structures to reduce tsunami power	
	Sanriku area	Flatland area	Sanriku area	Flatland area
Froude number F_r	0.5 to 0.65	0.8	> 0.9	> 1.27
Coefficient $a\ (=\sqrt{2}F_r)$	0.71 to 0.92	1.13	≥ 1.27	≥ 1.8

CONCLUSIONS

To verify the design load specified in the Japanese guidelines for tsunami shelters, the coefficient a is evaluated based on the damage observations after the 2011 Great East Japan Earthquake. The major findings can be summarized as follows:

1. The water depth coefficient a is larger than 1.7 in the areas without structures on the sea side that are deemed effective to reduce the tsunami power, while it is around 1 (or smaller especially when η_m is larger than 13m) in the areas with structures to reduce the tsunami power.
2. According to the discussion based on drag force, the tsunami pressure, the flow velocity, and the Froude number are more than 85kN/m^2, 9m/s, and 1.27 respectively, in the areas without structures to reduce the tsunami power. In the areas with structures to reduce the tsunami power, they lie in the range of 15 to 30kN/m^2, 4 to 6m/s, and 0.5 to 0.65 in Sanriku area and 0.8 in flatland area, respectively. The coefficient a computed based on the results above roughly correspond to the result of conclusion 1.
3. The coefficient a obtained in this study is much smaller than the results obtained from the damage observations after the 2004 Indian Ocean Tsunami. This is probably because structures investigated after the Indian Ocean Tsunami were located just close to the coastlines with no built environment to reduce tsunami power and high tsunami waves therefore directly attacked the structures.
4. It should be noted therefore that the design lateral load for tsunami shelters should be determined further considering evidences obtained in other events and experimental researches as well as those found in this study.

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