DESIGN LOAD EVALUATION FOR TSUNAMI SHELTERS BASED ON DAMAGE OBSERVATIONS AFTER INDIAN OCEAN TSUNAMI DISASTER DUE TO THE 2004 SUMATRA EARTHQUAKE

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ABSTRACT :

Tsunami shelters are of great importance to mitigate casualties by earthquake-induced killer waves, and the design guidelines for their practical design are recently developed by a task committee under the Japanese Cabinet Office, since great earthquakes significantly affecting coastal regions are expected to occur in the near future in Japan. Although they propose a practical design formula to calculate tsunami loads acting on shelters, it is derived primarily based on laboratory tests with scaled models but not on damage observations. It is therefore essential to examine the design loads through comparison between observed damage and structural strength. In December 2004, a huge scale Sumatra Earthquake caused extensive and catastrophic damage to 12 countries in the Indian Ocean. The author visited Sri Lanka and Thailand to survey structural damage due to tsunami, and investigated the relationship between damage to structures, lateral strengths computed based on their member properties, and observed tsunami heights. In the survey, 28 simple structures generally found in the affected coastal regions were investigated. The investigated results show that the design tsunami loads proposed in the guidelines are found rational to avoid serious damage but may not be conservative if the load amplification due to drifting debris is taken into account.

KEYWORDS : 2004 Sumatra Earthquake, tsunami shelter, design load, damage survey, inundation depth

1. INTRODUCTION

Mitigating damage due to tsunami as well as due to strong ground shaking is of highest priority to minimize loss of human lives and properties in areas along the coastline susceptible to tsunami hazard. Since great earthquakes such as Tokai Earthquake and Tonankai-Nankai Earthquake significantly affecting coastal regions are expected to occur in the near future in Japan, a task committee was set up under the Japanese Cabinet Office to discuss requirements and criteria to identify or design tsunami shelters and the design guidelines for tsunami shelters were proposed in 2005 (JCO 2005). The guidelines introduced an equation to compute tsunami loads expected to act on shelters constructed on coastlines, which is currently the only formula in Japan available for practically evaluating design tsunami loads for shelters. The equation was, however, developed primarily based on laboratory tests of 2-dimensional scaled model (Asakura et al. 2000) and has not yet been verified through damage observations after natural earthquake-induced tsunamis. It should also be noted that few damage investigations have been made focusing on quantitative evaluation of tsunami loads on building structures unlike that of seismic loads in the building engineering field. The author therefore made extensive damage surveys of structures that experienced the 2004 Indian Ocean Tsunami to investigate the relationship between their lateral resistance and observed damage, and to verify the appropriateness of the design equation. In this paper, the outline of damage surveys and investigated results on design tsunami loads is presented.

2. DAMAGE SURVEYS

2.1. Surveyed Areas

Damage surveys were made in (1)the northeast and south of Sri Lanka (Trincomalee, Galle, Matara, Hambantota etc.) on February 19 through 26, 2005 and (2)Phuket Island and Khao Lak of Thailand on March 9 through 13, 2005. Figure 1 shows the epicenter and surveyed areas. They are located about 1600 km and 500 km away from the epicenter, respectively, and have been little affected by ground shaking (Nakano 2007).

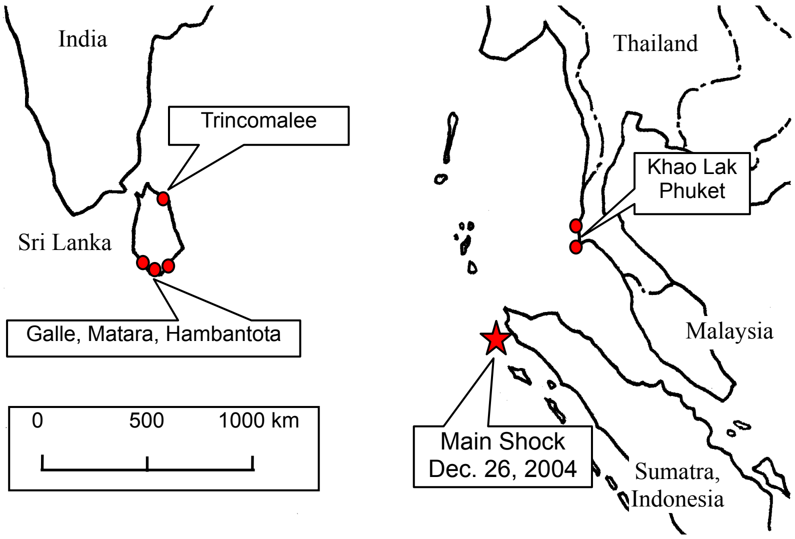

Figure 1 Epicenter and investigated areas

2.2. Survey Strategy

To collect as many damage data as possible for various types of structures and their structural properties, damage surveys were made at approximately 80 sites. Of the all surveyed structures, detailed surveys were made on 28 structures to record structural dimension and reinforcement arrangement to further investigate the relationship between their lateral resistance and tsunami load that acted on them since they met the following three conditions:

(1) The lateral resistance 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 trace height was clearly found on the surveyed site through water marks left on building's walls, where it was defined as the water depth above the ground level (i.e., inundation depth) at the structure's site. In addition to that, on-site interviews were also made to enrich tsunami inundation depth data if available.
(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.

Note that drifting debris as well as tsunami waves may have caused damaging impact on structures. Their effects were therefore considered in investigating the relationship between damage category and lateral resistance when the collision of debris was found to have obviously affected the damage to the surveyed structure.

2.3. Detailed Information Recorded on Surveyed Structures

Considering conditions for detailed surveys described earlier in 2.2, (a)building structures with simple configuration, (b)masonry (brick or concrete block) fence walls, (c)cantilever RC columns, (d)elevated water tanks supported by four columns, (e)Buddha's small mausoleums, and (f)small brick structures such as outhouses (i.e., outdoor toilets) and sheds were investigated for collecting detailed structural information. In the detailed surveys, the following data were collected at each site: (1)topographical information of the site, (2)maximum tsunami inundation depth obtained through measurement and, if necessary, supplementary on-site interviews, (3)building's use and structural type (RC, brick, concrete block, etc.), (4)damage category (no damage, cracked, or collapsed) and damage location(s), (5)structure and/or member dimension (B x D x H etc.), (6)reinforcement arrangement (diameter, spacing, cover concrete depth etc.), if it was an RC structure, and (7)general view photos and structural configurations of investigated structures.

Table 1 summarizes the investigated structures and photo 1 shows their typical damage patterns. Note that the structures categorized in (d) through (f) described above were generally found in the affected areas in Sri Lanka, and their data were collected to identify the criteria between damaged and survived structures even if they had minor or no damage. Detailed damage descriptions of surveyed structures and their structural information can be found in the related report (Nakano 2005).

3. EVALUATION OF LATERAL RESISTANCE OF INVESTIGATED STRUCTURES

According to the damage and failure mode observed, the flexural yielding strength M_y, the ultimate flexural strength at rebar fracture M_u, the overturning strength M_T, and the shear strength V_u are calculated, where M_y and M_u of RC members are computed from Eqs. (3.1) through (3.3) that are usually applied to beams and columns in Japanese design practice shown as follows:

$$M_y = 0.9\, a_t\, \sigma_y\, d \tag{3.1}$$
$$M_u = 0.9\, a_t\, \sigma_u\, d \tag{3.2}$$
$$M_y = 0.8\, a_t\, \sigma_y\, D + 0.5\, N\, D\, [\, 1 - N / (B\, D\, F_c)\,] \tag{3.3}$$

where M_y and M_u are the flexural yield strength and the ultimate flexural strength, respectively; σ_y and σ_u are the yield strength and the tensile strength of rebar, respectively; a_t is the cross-sectional area of tensile rebars; B, D, and d are the width, the depth, and the effective depth of a section, respectively; F_c is the compressive strength of concrete; and N is the axial load.

Note that most of columns investigated herein have low axial loads and their flexural resistance is evaluated from Eqs. (3.1) and (3.2) neglecting the axial load contribution to resistance while Eq. (3.3) is applied in calculating lateral resistance of a 2-story building designated by S53 in Table 1 (see also (8) in photo 1). It should also be noted that the factor 0.8 in Eq. (3.3) is modified according to the ratio of cover concrete to depth observed in the structure since the cover is thicker than the construction practice generally found in Japan. In computing the strength, the yield and tensile strength of reinforcing bars (σ_y and σ_u in Eqs. (3.1) through (3.3)) are determined from tensile tests of sample rebars (two samples from buildings in Sri Lanka and six samples from those in Thailand) that are carried out in Japan. The shear strength V_u of brick walls is defined as the product of its cross sectional area A_w in the principal direction of the structure along tsunami attack and the ultimate shear stress τ_u, where τ_u is assumed 0.4 N/mm^2 considering the wall's configuration and the brick's quality generally found in the affected areas. The contribution of walls in the direction perpendicular to the tsunami attack is neglected. In calculating the lateral resistance of 2-story building S53, the load-deformation relationship is assumed to reach its peak when the brick fails. The contribution of RC columns to the overall resistance is therefore reduced to half of their ultimate strength assuming the compatibility of deformation between stiffer brick and softer RC columns, which is consistent with the assumptions found in the Japanese Standard for Seismic Evaluation of Existing RC Buildings (JBDPA 2005).

Table 1 Strength, coefficient a, and damage of investigated structures (shaded rows denote column-shaped structures)

ID	Description of structures	Structure type*1	Location	My, Mu, M_T (kNm)	Vu (kN)	Inundation depth η_{max} (m) *2	Coeff. a *3	Damage*4	Remarks
S01	Column at entrance gate	RC	Trincomalee	My: 20	–	(3.0)	1.98	○ (×)	Possibly damaged by drifting debris
S06	Fence wall (1)	RC+B	Trincomalee	M_T: 50.5	–	0.9	1.73	×	Just on coastline
S08	Wall of outhouse (1)	B	Trincomalee	M_T: 35	–	1.3	2.38	×	Bond failure of mortar between brick units considered in M_T
S12	Fence columns	RC	Galle, cricket field	My: 6.6	–	3.0	1.13	×	Possibly damaged by drifting debris
S15	Columns supporting elevated water tank (2)	RC	Galle	M_T:104	–	2.4	3.03	(×)	Damaged by a drifting bus
S16	Cantilever columns of automobile factory's office	RC	Galle	My: 26	–	2.4	1.76	○	
				Mu: 33	–	2.4	2.04	○ (×)	No damage unless hit by a bus
S19	Fence wall (2)	RC+CB	Galle	Mu:135	–	2.35	1.01	×	CB walls neglected in Mu
S23	Wall of nursery school	B	Galle	–	349	1.55	4.03	(△)	Scratch found on wall; possibly damaged by drifting debris
S24	Wall of small shed (1)	B	Galle	–	480	1.6	>5.0	○	Located just behind S23
S25	Wall of outhouse (4)	B	Galle	–	130	1.6	3.68	○	Located just behind S23 / Entry perpendicular to tsunami direction
S26	Mausoleum of Buddha (3)	B	Galle	–	182	1.6	>5.0	○	Located just behind S23
S32	Columns supporting elevated water tank (3)	B	Hambantota	–	9.5	2.95	0.54	×	Bond failure of mortar between brick units considered in Vu
S33	Wall of outhouse (5)	B	Hambantota	–	83	0.95	4.07	○	Entry perpendicular to tsunami direction
S37	Columns supporting elevated water tank (4)	B	Hambantota	–	305	2.6	4.42	○	Far from coastline
S38	Columns supporting elevated water tank (5)	B	Hambantota	–	925	(5.0)	2.92	○	
S45	Wall of outhouse (7)	B	Kottegoda	–	170	(3.0)	2.55	○	Entry perpendicular to tsunami direction
S46	Wall of small shed (2)	B	Matara	–	263	2.05	3.93	○	Entry perpendicular to tsunami direction
S48	Wall of outhouse (9)	B	Matara	–	90	2.05	2.22	○	Entry perpendicular to tsunami direction
S53	School building	RC+B	Matara	–	1316	2.85	2.31	○	2-story RC building / Seismic capacity evaluation performed
S57	Cantilever columns (1) under construction	RC	Hikkaduwa	My: 18	–	(9.0)	0.54	×	Of all 15 columns, 8 totally collapsed and 7 heavily damaged
				Mu: 23	–	(9.0)	0.59	×	
T01	Wine cellar's wall	B	Patong Beach	–	680	1.75	>5.0	○	Approx. 100m away from coastline
T02	Kamala Beach H&R	RC+B	Kamala Beach	–	1473	3.95	(1.60)	○	Approx. 60m away from coastline (not plotted in Figure 4)
T07	Fence columns (1)	RC	Thap Lamu, Phang Nga Navy Base	My: 10	–	2.65	0.43	×	Pull-out of round rebars observed / Approx. 1km from coastline
T09	Fence columns (2)	RC		My: 4.5	–	2.65	1.14	×	Pull-out, yielding, and fracture of round rebars observed
				Mu: 6.3	–	2.65	1.39	×	/ Approx. 1km from coastline
T10	Columns of pier house (next to Navy Base)	RC	Thap Lamu, Phang Nga	My: 4.7	–	2.65	0.93	×	Yielding and fracture of round rebars observed / Just on coastline
				Mu: 6.6	–	2.65	1.05	×	
T13	Guest house of Khao Lak Merlin Resort Hotel	RC+B	Khao Lak, Phang Nga	–	317	4.23	1.16	△ ×	Brick wall (l=380cm) considered in Vu Of 11 guest houses, 9 collapsed or washed away
T15	RC columns of La Flora Khao Lak Hotel	RC	Bang Niang Phang Nga	My: 12.9	–	5.0	0.70	×	Just on coastline
				Mu: 19.9	–	5.0	0.87	×	
T17	Cantilever columns (3) under construction	RC	Phang Nga	My: 28.6	–	3.3	1.62	×	Far from coastline (estimated at more than 1 km away)
				Mu: 43.9	–	3.3	2.02	×	

*1 RC: RC columns, B: Unreinforced (UR) brick wall, CB: UR concrete block wall *2 Tsunami inundation depth above ground level (values in ( ) denote estimated depth after interviews)

*3 a = [computed water depth η' at Mu, My, M_T , or Vu] / [η_{max}] T02 is not plotted in Figure 4 due to uncertain information of reinforcing details.

*4 ○: no damage, △: cracked, ×: collapsed or extensively tilted, ( ): damage due to drifting debris (more than a single mark at an identical plot in Figure 4 denotes coexistence of different types of failure)

4. COMPARISON BETWEEN TSUNAMI LOAD AND OBSERVED DAMAGE

In the guidelines, the design tsunami load is defined by Eq. (4.1). In the subsequent investigations, Eq. (4.2) that is analogous to Eq. (4.1) is first defined, and the coefficient a is evaluated setting the lateral resistance of investigated structure equal to the tsunami load computed from Eq. (4.2):

$$q_x(z) = \rho\, g\, (\, 3\, h - z\,) \tag{4.1}$$
$$p_x(z) = \rho\, g\, (\, a\, \eta_{\max} - z\,) \tag{4.2}$$

where $q_x(z)$ (kN/m^2) is the design tsunami pressure acting on a structure at a distance z above the ground level

(1) S01: RC columns (2) S08: Brick outhouse (3) S16: Cantilever RC columns with rebar fracture

(4) S19: CB fence wall w/ RC column (5) S25: Brick outhouse (6) S26: Mausoleum (7) S32: Elevated water tank

(8) S53: 2-story RC school (9) S57: RC columns under construction (rebars bent and some columns failed)

(10) T09: RC fence columns of Navy Base (rebar fractured) (11) T15: RC columns under construction

Photo 1 Typical damage to investigated structures ("S01" etc: ID Nos. in Table 1, "→": tsunami flow direction) 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, z (m) is the distance above the ground level to compute tsunami pressure p_x and q_x ($0 \leq z \leq 3h$ for Eq. (4.1) and $0 \leq z \leq a\ \eta_{max}$ for Eq. (4.2)), $p_x(z)$ (kN/m^2) is the tsunami pressure acting on a structure at a distance z above the ground level where η_{max} (m) is the observed tsunami inundation depth, a is the ratio of the water depth η' equivalent to structure's ultimate strength to the observed tsunami inundation depth η_{max} (i.e., $a = \eta'/\eta_{max}$). Note that the inundation depth for h and η_{max} is defined as the water depth above the ground level at the building's location.

Figure 2 illustrates the background concept employed in Eq. (4.1). 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), which is based on the laboratory tests of 2-dimensional scaled model (Asakura et al., 2000). To examine whether or not the coefficient 3 in Eq. (4.1) is appropriate to evaluate the tsunami load, Eq. (4.2) is introduced in the manner analogous to Eq. (4.1). If the coefficient a successfully categorizes damaged and survived structures at its value of 3, one can say that Eq. (4.1) with a equal to 3 is a rational design formula to compute the tsunami load effect. 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 coefficient a can be computed assuming that the lateral resistance of an investigated structure is equal to the overall tsunami load acting on it under the pressure distribution along its height defined by Eq. (4.2). The

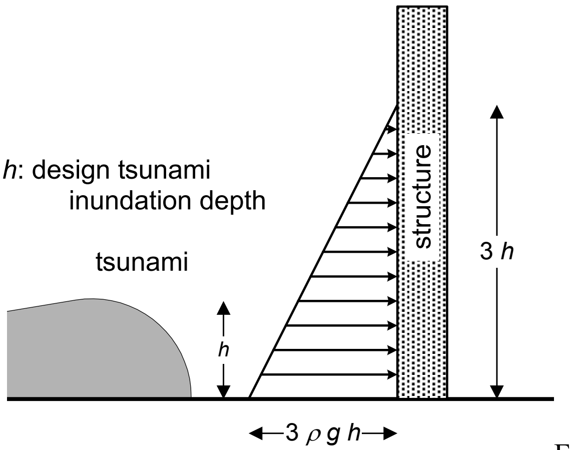

Figure 2 Design tsunami pressure distribution (JCO 2005)

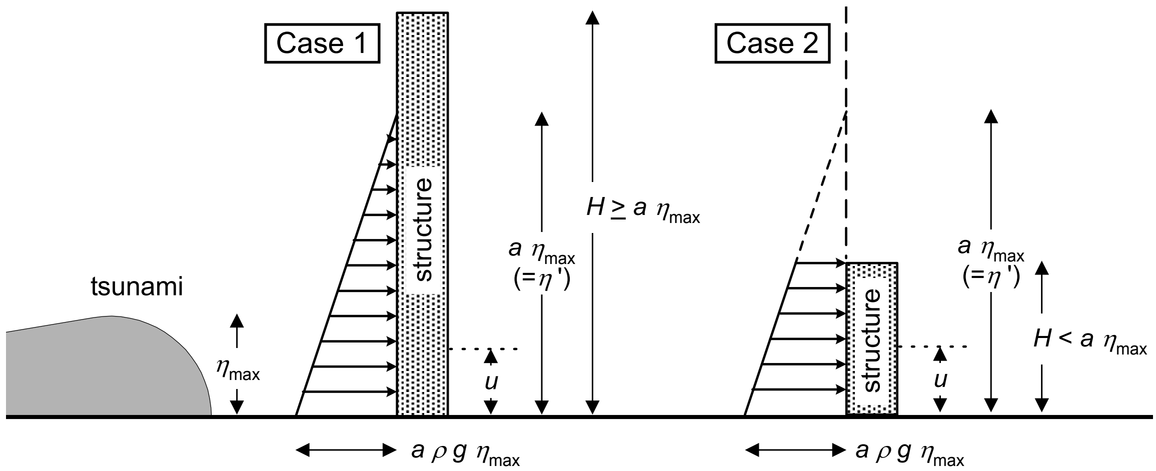

Figure 3 Tsunami inundation depth η_{max}, building height H, and tsunami pressure distribution p_x (Nakano 2007)

coefficient therefore denotes the ratio of equivalent water depth η' corresponding to the structure's lateral resistance under a triangular hydrostatic pressure profile to the observed inundation depth η_{max}. The procedure to compute the coefficient a is described in detail below.

1. Compute the lateral resistance of investigated structures considering their failure mode as shown earlier in Section 3.
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 Eq. (4.2). Setting the force or moment at the height u equal to the lateral resistance obtained in step 1. above, evaluate the equivalent water depth η' corresponding to the resistance. Note that the tsunami pressure above structures is neglected and the depth η' 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 η' to observed tsunami inundation depth η_{max} (i.e., $a = \eta'/\eta_{max}$).

Table 1 shows the investigated tsunami inundation depth η_{max} and the computed coefficient a. Their relationship is shown in Figure 4(a) for wall-shaped structures such as fence walls and in Figure 4(b) for column-shaped structures such as cantilever RC columns, respectively, where the structure type is determined based on the shape of member on which the tsunami attacks. When the structures with identical structural properties have different failure patterns due to the effects of drifting debris or some other reasons, two marks corresponding to different failure patterns are plotted at the same point of the figure.

Figure 4(a) shows that structures with the value of a greater than 2.5 have no major damage except for the case S23 that may have been damaged due to drifting debris, and the value of 3 for the coefficient a proposed in the guidelines can be considered rational to avoid serious damage due to tsunami attack. It should be noted, however, that the structure (S23) having the coefficient a greater than 4 suffers wall cracking, and the coefficient of 3 may not be conservative if the load amplification due to drifting debris is taken into account, and countermeasures to protect structures from damage due to drifting debris need to be taken.

Figure 4(b) shows that the coefficient a for column-shaped structures to discriminate between damaged and survived may lie at around 2 when the effects of drifting debris are neglected, which is slightly lower than that for wall-shaped structures. This result implies that column-shaped structures have advantage in tsunami

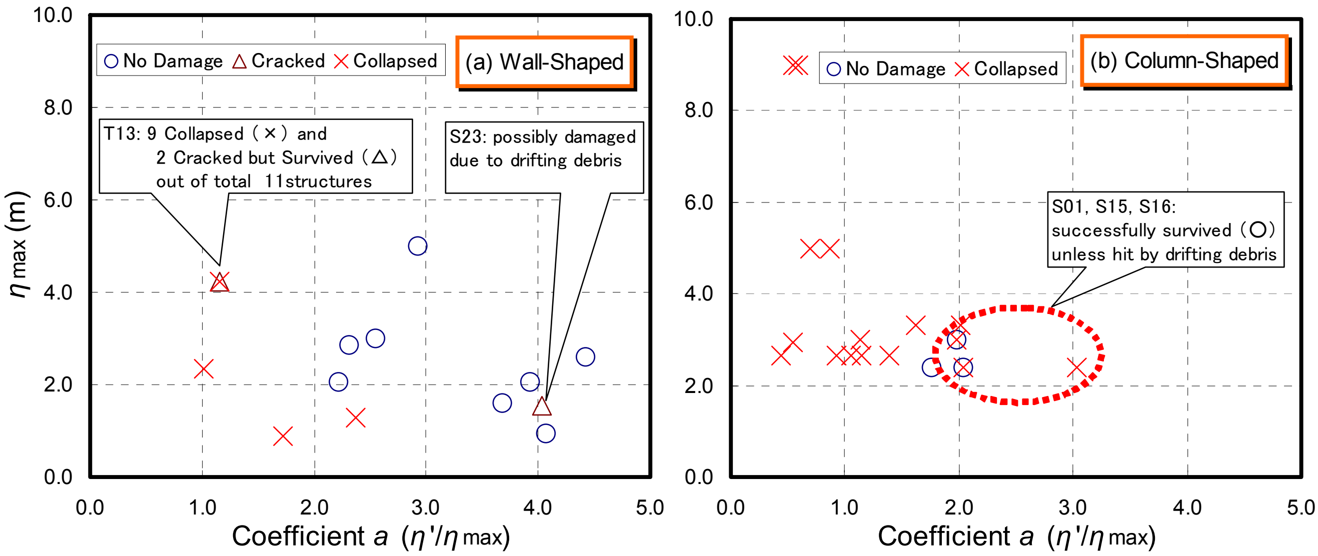

Figure 4 Computed Coefficient a vs. observed tsunami inundation depth η_{max}
(Numerals in the figure denote ID Nos. in Table 1)

resisting performance over wall-shaped structures on condition that both structures have enough seismic capacity to survive the ground shaking prior to the tsunami attack. It should also be noted, however, that the column-shaped structures can not be left undamaged at the coefficient a in the range of 2 to 3 as shown for the cases of S01, S15, and S16, and countermeasures against drifting debris need to be taken to protect structures as is the case of wall-shaped structures previously described.

5. CONCLUSIONS

To examine the design load specified in the Japanese guidelines for tsunami shelters, damage surveys are made in Sri Lanka and Thailand after the 2004 Indian Ocean Tsunami disaster, and the lateral strengths of structures in the affected areas, the tsunami load computed by the design formula considering tsunami inundation depth, and the observed damage are mutually compared. The major findings can be summarized as follows:

1. The value of coefficient 3 for computing design tsunami loads proposed in Eq. (4.1) of the guidelines compares well with the criteria between damaged and survived structures in the tsunami affected areas surveyed after the 2004 Indian Ocean Tsunami disaster, and the design tsunami load specified in the guidelines is found rational.
2. The value, however, may not be conservative if the load amplification due to drifting debris is taken into account, and other countermeasures would be needed to avoid unexpected damage due to debris.

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REFERENCES

Asakura, R., Iwase, K., Ikeya, T. Takao, M., Kaneto, T., Fujii, N., Omori, M. (2000). An Experimental Study on Wave Force Acting on On-Shore Structures due to Overflowing Tsunamis, *Proceedings of Coastal Engineering*, Japan Society of Civil Engineers, **47**, 911-915.

JBDPA/Japan Building Disaster Prevention Association (2005). Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings, 2001; Guidelines for Seismic Retrofit of Existing Reinforced Concrete of Existing Reinforced Concrete Buildings, 2001; Technical Manual for Seismic Evaluation and Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001, English Version, **1st**.

JCO/Task Committee under the Japanese Cabinet Office (2005). Design Guidelines for Tsunami Shelters (in Japanese), http://www.bousai.go.jp/oshirase/h17/tsunami_siryo2.pdf

Nakano, Y. (2005). Damage Report on Structures due to 2004.12.26 Sumatra Earthquake Ver. 2.2, http://sismo.iis.u-tokyo.ac.jp/Research.files/topic4.files/topic4-007.files/T4-7-1.pdf

Nakano, Y. (2007). DESIGN LOAD EVALUATION FOR TSUNAMI SHELTERS BASED ON DAMAGE OBSERVATIONS AFTER INDIAN OCEAN TSUNAMI DISASTER DUE TO THE 2004 SUMATRA EARTHQUAKE, *Journal of Architecture and Building Science*, Architectural Institute of Japan, **13: 25**, 337-340.