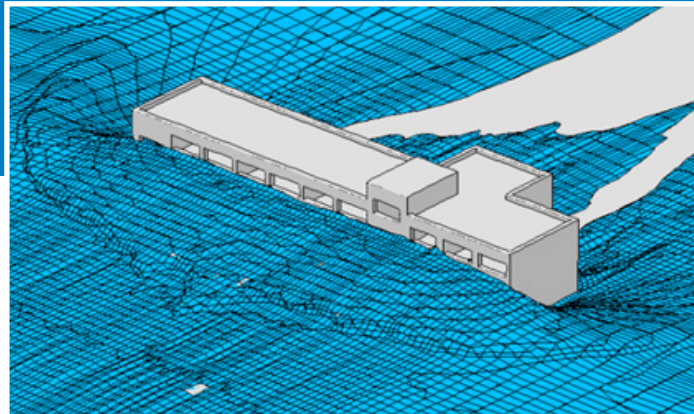


An ACI Technical Publication

SYMPOSIUM VOLUME



Proceedings of the First ACI & JCI
Joint Seminar: Design of Concrete
Structures Against Earthquake and
Tsunami Disasters

Editors:
Kyuichi Maruyama and Andrew W. Taylor

SP-313



American Concrete Institute
Always advancing



Documents

Document Sharing Hub

Proceedings of the First ACI & JCI Joint Seminar: Design of Concrete Structures Against Earthquake and Tsunami Disasters

Editors:
Kyuichi Maruyama and
Andrew W. Taylor



American Concrete Institute

Always advancing

SP-313

First printing, March 2017

Discussion is welcomed for all materials published in this issue and will appear ten months from this journal's date if the discussion is received within four months of the paper's print publication. Discussion of material received after specified dates will be considered individually for publication or private response. ACI Standards published in ACI Journals for public comment have discussion due dates printed with the Standard.

The Institute is not responsible for the statements or opinions expressed in its publications. Institute publications are not able to, nor intended to, supplant individual training, responsibility, or judgment of the user, or the supplier, of the information presented.

The papers in this volume have been reviewed under Institute publication procedures by individuals expert in the subject areas of the papers.

Copyright © 2017
AMERICAN CONCRETE INSTITUTE
38800 Country Club Dr.
Farmington Hills, Michigan 48331

All rights reserved, including rights of reproduction and use in any form or by any means, including the making of copies by any photo process, or by any electronic or mechanical device, printed or written or oral, or recording for sound or visual reproduction or for use in any knowledge or retrieval system or device, unless permission in writing is obtained from the copyright proprietors.

Printed in the United States of America

Editorial production: Gail Tatum

ISBN-13: 978-1-945487-55-2

Preface

The First American Concrete Institute (ACI) and Japan Concrete Institute (JCI) joint seminar was conceived as a vehicle for promoting collaboration and cooperation between two organizations that are dedicated to the global advancement of concrete technology. In September 2012 ACI President James Wight, and ACI Executive Vice President Ronald Burg, visited the headquarters of JCI and discussed ways to promote collaboration between ACI and JCI with JCI President Taketo Uomoto and JCI Executive Directors. A joint ACI and JCI technical seminar was proposed as a way to share knowledge and foster collaboration between the two organizations. Subsequent discussions between Ronald Burg and JCI Executive Director Kyuichi Maruyama led to a joint seminar planning meeting, held at the ACI convention in Minneapolis, Minnesota, in April 2013.

This volume contains the technical papers presented at the First ACI & JCI Joint Seminar, held in Waimea, Island of Hawaii, Hawaii, July 16 to 18, 2014. The theme of the joint seminar was “Design of Concrete Structures Against Earthquake and Tsunami Disasters.” Five papers were presented by authors from ACI, and five papers from JCI. Three papers are related to tsunami loads and structural design requirements, and seven are related to seismic analysis and design.

The three papers on tsunami effects included a summary by Nakano of structural design requirements for tsunami evacuation buildings in Japan; an overview by Chock of the new tsunami load and design requirements in the United States; and a study by Maruyama et al. on the evaluation of tsunami forces acting on bridge girders.

The seven papers on seismic effects addressed topics ranging from seismic design standards to innovative methods of construction for seismic retrofit. Parra-Montesinos et al. presented the results of experiments on fiber-reinforced coupling beams, as well as design guidelines. Teshigawara discussed JCI contributions to the ISO Standard for seismic evaluation and retrofit of existing concrete structures. A summary of a project on the use of high-strength reinforcement for seismic design was presented by Kelly et al., including findings that are based on extensive prior research on high-strength reinforcement in Japan. Shiohara described the results of a study that supports the new Architectural Institute of Japan (AIJ) Standard for Seismic Capacity Calculation, with a focus on beam-column joints and collapse simulation. Matamoros presented a study of factors that affect drift ratio at axial failure of nonductile reinforced concrete buildings. A study of the seismic response of reinforced concrete bridge piers, including the effects of interaction between piles and soil, was presented by Maki et al. Finally, French et al. discussed an overview of lessons learned from laboratory testing of reinforced concrete shear walls.

The day after the joint seminar a meeting was held between ACI and JCI officials to discuss future collaboration and joint seminars. Representing ACI were President William E. Rushing, and the ACI Executive Vice President, Ronald Burg. Representing JCI were President Hirozo Mihashi, and Chair of the JCI Committee on JCI-ACI Collaboration, Kyuichi Maruyama. It was resolved to hold a second joint seminar, to be hosted by JCI in Tokyo, in conjunction with the 50th anniversary celebrations of the founding of JCI on July 13, 2015. In addition, subsequent discussions between ACI and JCI led to plans for the third joint seminar, to be hosted by ACI at the ACI Convention in Anaheim, California, in October 2017.

It is hoped that this collection of papers will serve to advance the state of analysis and design of concrete structures against earthquakes and tsunamis in both the United States and Japan, and that it will serve as a model for future collaboration between ACI and JCI.

Kyuichi Maruyama
JCI Co-Editor

Andrew W. Taylor
ACI Co-Editor

TABLE OF CONTENTS

SP-313—1

Structural Design Requirements for Tsunami Evacuation Buildings in Japan..... 1.1

Author: Yoshiaki Nakano

SP-313—2

The ASCE 7 Tsunami Loads and Effects Design Standard for the U.S. 2.1

Author: Gary Chock

SP-313—3

Evaluation of Tsunami Force Acting on Bridge Girders 3.1

Authors: Kyuichi Maruyama, Yasushi Tanaka, Kenji Kosa, Akira Hosoda, Taro Arikawa, Norimi Mizutani, and Tomoaki Nakamura

SP-313—4

Elimination of Diagonal Reinforcement in Earthquake-Resistant Coupling Beams through Use of Fiber-Reinforced Concrete.....4.1

Authors: G.J. Parra-Montesinos, J.K. Wight, C. Kopczynski, R.D. Lequesne, M. Setkit, A. Conforti, and J. Ferzli

SP-313—5

JCI Activity on ISO Standard for Seismic Evaluation of Concrete Structures 5.1

Author: Masaomi Teshigawara

SP-313—6

ATC-98 Project on Seismic Design of Concrete Structures with High-Strength Reinforcement..... 6.1

Authors: Dominic J. Kelly, Andres Lepage, David Mar, Jose I. Restrepo, Joseph C. Sanders, and Andrew W. Taylor

SP-313—7

A New AIJ standard for Seismic Capacity Calculation: Recent Advances in Beam-Column Joint Design and Seismic Collapse Simulation on Reinforced Concrete Frame Buildings 7.1

Author: Hitoshi Shiohara

SP-313—8

Factors that Affect the Drift at Axial Load Failure of Nonductile RC Buildings 8.1

Author: Adolfo Matamoros

SP-313—9

Seismic Response of RC Bridge Piers Considering Soil-Pile Interaction 9.1

Authors: Takeshi Maki, Hiroshi Mutsuyoshi and Anawat Chotesuwan

SP-313—10

Lessons Learned from Reinforced Concrete Wall Tests 10.1

Authors: Catherine E. French, Beth Brueggen, Sri Sritharan, Sriram R. Aaleti, Suzanne Dow Nakaki

Structural Design Requirements for Tsunami Evacuation Buildings in Japan

Yoshiaki Nakano¹

Synopsis: The Great East Japan Earthquake that struck northern Japan in March 2011 caused devastating tsunami damage, both to property and human life. To evacuate inland or to elevated ground is the primary action immediately to be taken in coastal areas after a felt earthquake. But there are plenty of communities where people simply cannot evacuate in time, and constructing tsunami evacuation buildings at strategic locations is therefore vital means to effectively mitigate human damage. After the 2011 catastrophic tsunami event, a joint team of the Institute of Industrial Science, The University of Tokyo (IIS UTokyo) and the Building Research Institute (BRI) extensively inspected tsunami damaged buildings and investigated their lateral strength, structural type, site condition, observed damage etc. In November 2011, The Ministry of Land, Infrastructure, Transport and Tourism newly issued the Interim Guidelines on the Structural Design of Tsunami Evacuation Buildings considering new findings, improved knowledge, and various experiences learned through the repeated damage investigations (Guidelines 2011). This paper presents the outline of the structural requirements for tsunami evacuation buildings stipulated in the new Japanese Interim Guidelines 2011. Following the Guidelines 2011, the relationship between structural size, required lateral strength, and tsunami inundation depth is also studied and discussed herein.

Keywords: Tsunami, design, evacuation building, structural requirement

INTRODUCTION

The Great East Japan Earthquake that struck northern Japan in March 2011 caused devastating tsunami damage, both to property and human life. To evacuate inland or to elevated ground is the primary action immediately to be taken in coastal areas after a felt earthquake. But there are plenty of communities where people simply cannot evacuate in time, and constructing tsunami evacuation buildings at strategic locations is therefore vital means to effectively mitigate human damage.

To design and construct buildings resistive to tsunami loads, quantitative evaluations of tsunami load applicable to structural design is most essential. Since great earthquakes such as Tokai Earthquake and Tonankai-Nankai Earthquake significantly affecting coastal areas are expected to occur in the near future in Japan, the Central Disaster Prevention Council issued the General Principles for Countermeasures against Tokai Earthquake in May 2003 and against Tonankai-Nankai Earthquake in December 2003, respectively. Under such circumstances, the Building Center of Japan (BCJ) started a research project to discuss structural requirements for tsunami evacuation buildings and drafted a technical guide for their structural design in 2004 (BCJ Guidelines 2004) (Okada et al. 2004a and 2004b), which for the first time in Japan introduced a formula to compute tsunami loads expected to act on buildings and other structural requirements. The formula was developed primarily based on laboratory tests of 2-dimensional scaled models (Asakura et al., 2000) and examined through surveys of structures after the Indian Ocean Tsunami in December 2004 (Nakano 2007&2008). The Japanese Cabinet Office also set up a task committee to discuss requirements and criteria to design tsunami evacuation buildings and proposed design guidelines in 2005 (JCO Guidelines 2005) referring BCJ Guidelines 2004 mentioned above. However, few buildings had been designed based on these guidelines until 2011.

After the 2011 catastrophic tsunami event, a joint team of the Institute of Industrial Science, The University of Tokyo (IIS UTokyo) and the Building Research Institute (BRI) extensively inspected tsunami damaged buildings and investigated their lateral strength, structural type, site condition,

¹ Director General, Professor, Institute of Industrial Science, The University of Tokyo, Tokyo, Japan

observed damage etc. to review and verify the tsunami forces and to enrich design requirements and commentary described in the previous guidelines. The joint team proposed necessary revisions based on new findings, improved knowledge, and various experiences learned through the repeated damage investigations followed by intensive discussions. In November 2011, The Ministry of Land, Infrastructure, Transport and Tourism adopted the proposal and newly issued the Interim Guidelines 2011 on the Structural Design of Tsunami Evacuation Buildings (Guidelines 2011).

This paper presents the outline of the structural requirements for tsunami evacuation buildings stipulated in the new Japanese Interim Guidelines 2011. Following the Guidelines 2011, the relationship among structural size, required lateral strength, and tsunami inundation depth is also studied and discussed herein.

DESIGN PRINCIPLES AND INTERIM GUIDELINES 2011

The tsunami evacuation buildings can be affected by tsunamis from two sources, i.e., near-source-generated tsunamis and far-source-generated tsunamis. The first tsunami source would be the one that occurs following so called near-field earthquakes, which also would create severe and damaging ground shaking. The second source would be a distant earthquake that occurs far away from the coastal areas without any local earthquake effects. Following the 2011 event, the Japanese central and local governments have conducted extensive tsunami simulations using improved scientific data and methods, as well as considering inundated areas due to recent and historic tsunami events, to provide rational tsunami hazard maps which are of primary help for earthquake and tsunami disaster mitigation planning. As will be described later, the tsunami loads are in general given considering the tsunami inundation depth that appears in the regional hazard maps provided by the local government.

The tsunami evacuation buildings are primarily required to have the capacity to resist anticipated tsunami loads without collapse, overturning, or lateral movement for the life safety of evacuees. Figure 1 shows a basic flow of structural design procedure. As was often found after the 2011 tsunami disaster, the performance of a building during tsunami inundation is significantly affected by buoyancy in addition to the tsunami flow, and uplift due to buoyancy should be carefully taken into account in the design. The structural safety and integrity is the primary concern for the tsunami evacuation building, but it should also be noted that the refuge areas should be located well above the elevation considering possible splash-up during tsunami impact and inherent uncertainty in estimating tsunami run-up elevation.

In designing individual members, they are first categorized as either breakaway or non-breakaway components. Breakaway components are allowed to fail under a specific tsunami load without causing damage to the building system. This concept is employed because it is deemed impractical to design all members and their connections strong enough to resist the maximum considered tsunami event from technical and economical point of view. Structural components, however, should be designed as non-breakaway to resist and transfer the forces acting on them.

Tsunami debris impacts and scour are also essential issues in designing tsunami evacuation buildings. When tsunamis propagate inland, destructive waves can carry debris creating high impact loads and cause extensive damage to timber houses although they may generally cause local damage to reinforced concrete buildings. Due to uncertainties involved in the estimate of impact forces associated with waterborne debris, the Guidelines 2011 incorporate considerations of the *missing column strategy*, which also appears in the Japanese Seismic Evaluation Standard (JBDPA 1977), to reduce the potential for progressive collapse if one column is severely damaged and loses its vertical load carrying capacity.

Tsunami scour depth is generally difficult to predict because of the many variables that govern the scour mechanism. Deep concrete foundations (pile foundations) should be provided for the tsunami evacuation buildings instead of shallow foundations such as mat foundations, because of the scour

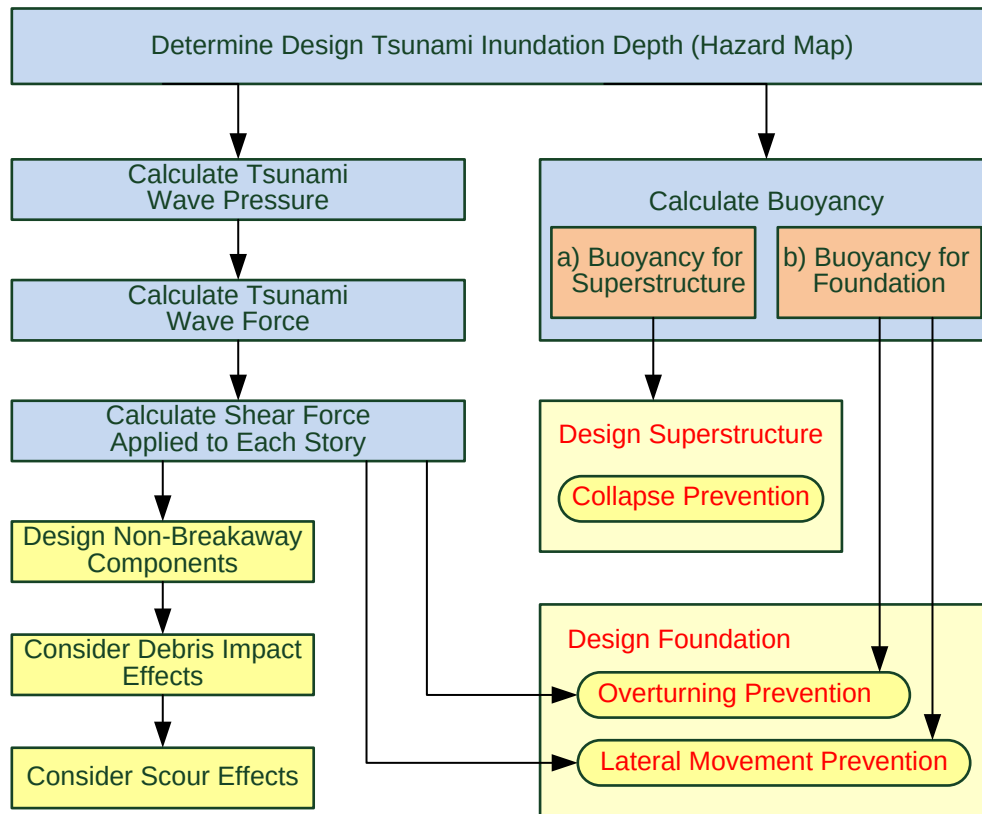


Figure 1. General structural design procedure. (after NILIM 2012)

potential that can occur during a tsunami.

KEY ISSUES FOR STRUCTURAL DESIGN OF TSUNAMI EVACUATION BUILDINGS

Tsunami wave pressure and its profile

It is well accepted that the tsunami wave pressure acting on structures and its profile are complicatedly dependent on the tsunami inundation depth, velocity, water flow to and around the structures, etc.

Tsunami evacuation buildings may be often constructed inland where complicated effects on the buildings may be caused by existing structures, and the tsunami velocity at each construction site, therefore, is not necessarily predicted with a reliable value due to local effects. In addition, the tsunami inundation depth shown in tsunami hazard maps provided by the local governments is the primary and in general the only source currently available to determine the tsunami loads in Japan.

Considering the current state of practice and simplicity point of view, the tsunami pressure profile is tentatively defined in the form of equivalent hydrostatic pressure as shown in Eq. (1), which is in the analogous form shown in the previous BCJ Guidelines 2004 and JCO Guidelines 2005. In Eq. (1), a water depth coefficient a is employed and its value can be any one of 1.5, 2.0, or 3.0. The value of coefficient a is primarily 3.0 unless tsunami energy dissipation is expected, which is based on the 2-dimensional hydraulic test results of scaled model (Asakura et al., 2000) to simulate and evaluate tsunami loads acting on inland buildings.

Figure 2 illustrates the background concept employed in Eq. (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., $a = 3.0$ in Eq. (1)) and the pressure at the

bottom is assumed 3 times of the hydrostatic pressure. The equation form shows that the influence of water velocity is implicitly incorporated in the coefficient a larger than 1.0. The coefficient a is also investigated through the relationship between observed damage and structural resistance found in coastal areas after the Indian Ocean Tsunami in 2004, and the value of 3.0 is found rational to avoid serious damage to structures unless hit by tsunami debris.

$$q_z = \rho g (a h - z) \quad (1)$$

where,

q_z : intensity of tsunami pressure at height z (kN/m²),

ρ : density of water (1.0 t/m³ assumed herein),

g : gravitational acceleration, 9.8 (m/s²),

a : water depth coefficient. The value of a is primarily 3.0 but can be reduced if the building is located in the condition shown in Table 1 and Figure 3.

h : design inundation depth (m),

z : location of acting pressure measured from the ground ($0 \leq z \leq a h$) (m).

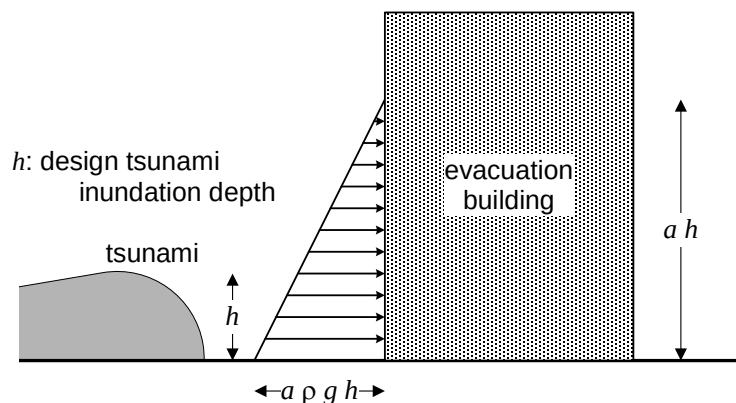


Figure 2. Design tsunami pressure distribution.

Following the 2011 event, damage surveys have been made by various institutes and research organizations. IIS UTokyo and BRI including the author also made extensive damage investigations to verify the value of a in the analogous way employed in surveys after the 2004 Indian Ocean Tsunami (Nakano, 2007&2008), and found the value is smaller than those obtained after the Indian Ocean Tsunami. This is probably because structures investigated after the Indian Ocean Tsunami were located just close to the shorelines without any coastal structures such as seawalls, bulkheads and revetments to dissipate tsunami energy and high tsunami waves therefore directly attacked the structures. In determining design values, however, it is also essential to carefully consider uncertainties associated with natural hazard, local effects due to building location as well as evidences found in experiments (Asakura, 2000) and field surveys such as Indian Ocean Tsunami (Nakano 2007&2008). The primary value of coefficient a in the Guideline 2011 is therefore determined 3.0 while it can be reduced to 2.0 or 1.5 due to the presence of tsunami energy dissipation or deflection structures and the distance from the shoreline. In reducing the coefficient a , the following two evidences found after the 2011 event are considered.

- (1) The coefficient a for buildings with energy dissipation or deflection structures (either onshore or offshore structures) could be roughly 1/1.5 of that for those without such structures.
- (2) In areas where the distance from a shoreline or riverbank is farther than 500m, the coefficient a could be around or less than 1 although the limited number of data. It should also be noted that the observed tsunami inundation depth has wide scatterness around the simulated values and is often as high as 1.5 times of simulations.

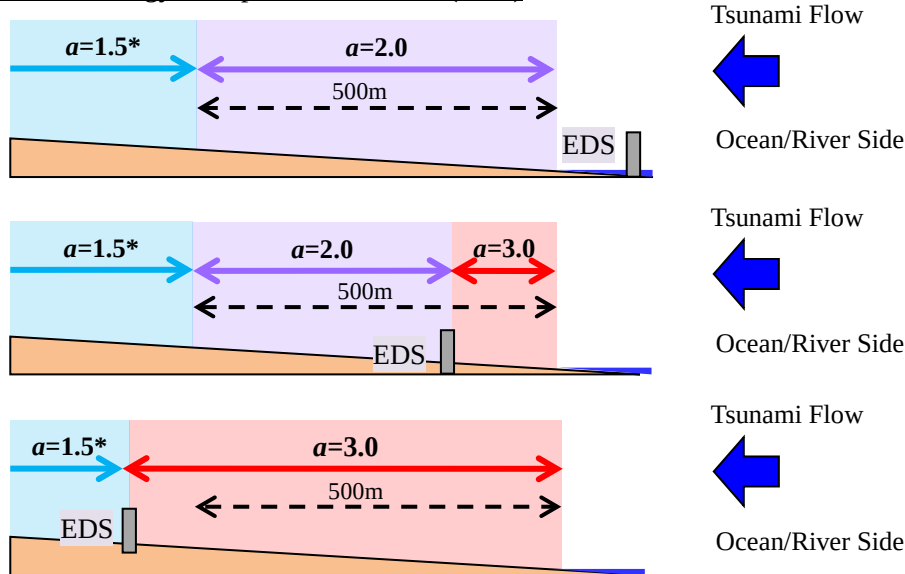
In the Guidelines 2011, the primary value of 3.0 for the coefficient a is reduced to 2.0 (= 3.0 / 1.5)

when tsunami energy dissipation or deflection structures facing the ocean are provided and to 1.5 (= 1.0×1.5) when the building is located 500m or farther from the shoreline or riverbank. Table 1 and Figure 3 summarize the proposed value of coefficient a in the design equation Eq. (1).

Table 1. Water depth coefficient a .

Energy Dissipation Structures Provided		No Energy Dissipation Structures (at any distance from shoreline or riverbank)
Distance from shoreline or riverbank		
Distance $\geq 500\text{m}$	Distance $< 500\text{m}$	
1.5	2.0	3.0

Areas within Energy Dissipation Structures (EDS)



Areas without Energy Dissipation Structures (EDS)

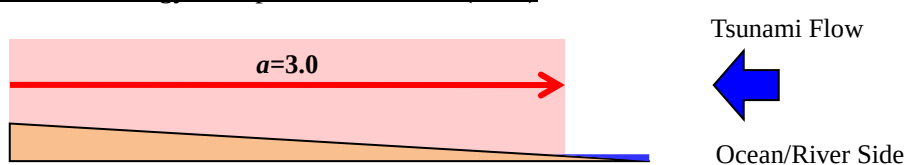


Figure 3. Water depth coefficient a considering energy dissipation structures and distance from shoreline or riverbank. (after IIS UTokyo 2011)

Tsunami force acting on components

The tsunami wave force can be calculated by integrating its acting pressure shown in Eq. (2) considering the pressure-exposed surface area and the pressure distribution along the height of the building concerned. In calculating the force acting on a building, the contribution of nonstructural components such as standard residential entry doors, shutter doors, windows on an exterior frame, which are expected to fail during the early phase of tsunami exposure and therefore considered *openings*, can be neglected. It should be noted, however, that the reduced force acting on an exterior frame should not be less than 70% of that without such openings because the presence of interior structural walls and other members may cause less effective response in reducing the tsunami force on a building. Figure 4 shows numerical simulation results on the relationship between force reduction and opening ratio. It shows that the force acting on the model structures can be reduced with increase in its opening ratio but the reduction may have a lower bound value of around 70% due to interior wall contribution except for an elevated building.

$$Q_z = \rho g \int_{z_1}^{z_2} (a h - z) B dz \quad (2)$$

where,

Q_z : design tsunami wave force (kN),

B : width of pressure-exposed surface (m),

z_1 : minimum height of pressure-exposed surface ($0 \leq z_1 \leq z_2$) (m),

z_2 : maximum height of pressure-exposed surface ($z_1 \leq z_2 \leq a h$) (m).

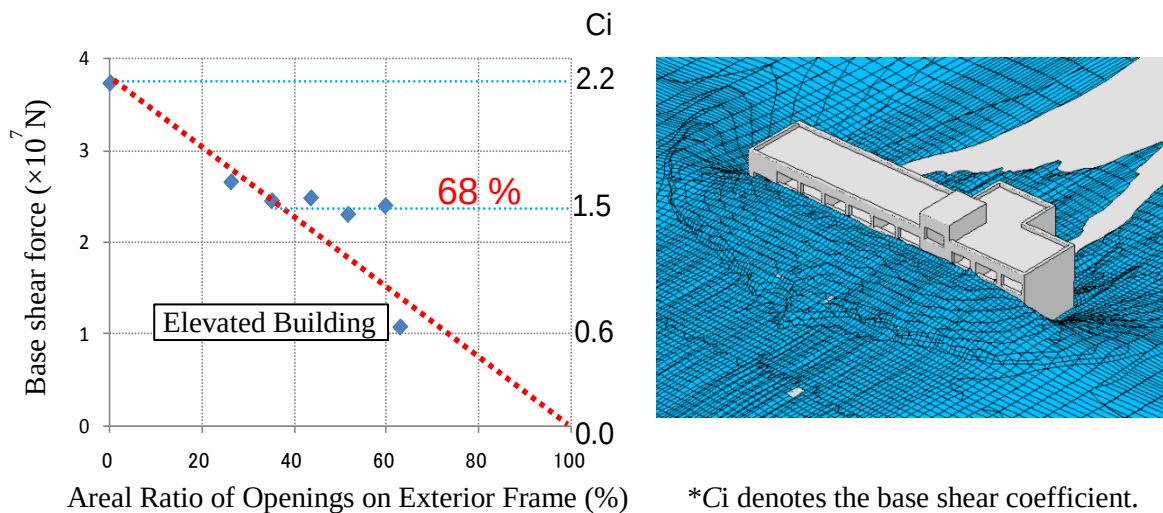


Figure 4. Relationship between base shear force and areal ratio of openings. (Okuda et al. 2008)

This limit of 70%, therefore, does not apply to columns provided in the open space of a piloti style building, i.e., a building raised on columns, to allow for tsunami passage beneath the building. It should be noted, however, that such an elevated building can significantly increase the potential for the ground shaking damage, and should be most carefully designed and constructed to properly resist damaging ground shaking.

The force calculated above is applied to confirm the safety of a whole building against collapse, overturning, and lateral movement as well as of non-breakaway components. To examine the force vs. displacement performance of a whole building to the expected tsunami event, a pushover analysis is generally performed using the force distribution defined in Eq. (1). It should be noted that tsunami waves have much longer periods than earthquake shakings and tsunami evacuation buildings should

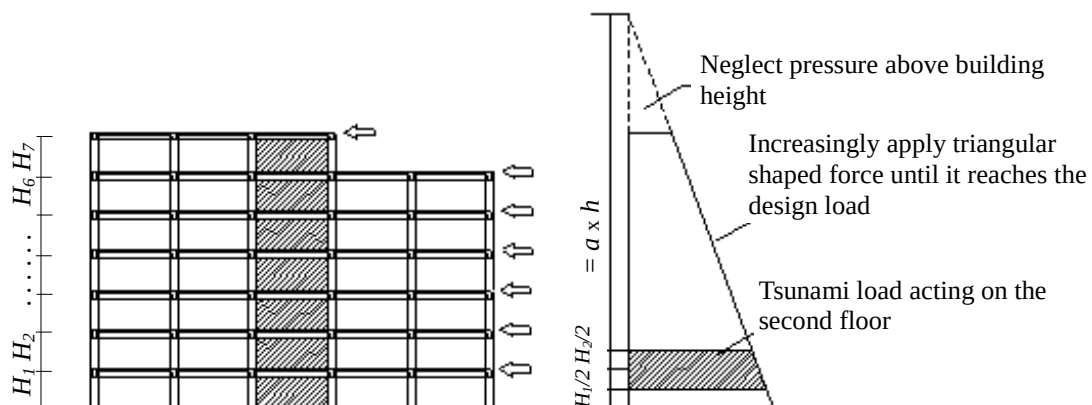


Figure 5. Tsunami load applied to each floor in pushover analyses. (after NILIM 2012)

properly resist such tsunami-induced forces. Figure 5 shows an example of tsunami loads applied to a building during pushover analyses. Unlike general seismic design, tsunami evacuation buildings therefore need to be more emphasized on their resistance rather than ductility due to the differences in shaking and tsunami characteristics.

Buoyancy effects

Uplift due to buoyancy has the effect of reducing the total weight of a building, which may significantly impact the resistance for overturning and lateral movement. After the 2011 tsunami event, 8 overturned buildings are investigated and found to have a small opening ratio of less than 0.3. Rising water will pose significant effects on uplifting and overturning of such building during rapid inundation. Furthermore the reduction of the weight also leads to the reduction of lateral strength of structural members.

Buoyancy is affected by the volume of water flowing into a building but it may be generally difficult to precisely predict the water inflow during a tsunami inundation in the design stage. In addition to the volume of displaced water of a submerged building and its components, the additional volume of water displaced by air trapped below the floor framing system, which was very often found in the inundated buildings after 2011 event, can significantly contribute to the buoyancy. Considering the average unit weights of each floor in steel buildings and reinforced concrete buildings in Japan are 8 kN/m^2 and 13 kN/m^2 , respectively, the excluded water height of 0.8 m and 1.3 m in each story of a box-type building may lose the downward resistance by self-weight, and uplift forces from buoyancy therefore should be carefully taken into account. In the Guidelines 2011, the following assumptions as shown in Figure 6 are employed for the safe side design.

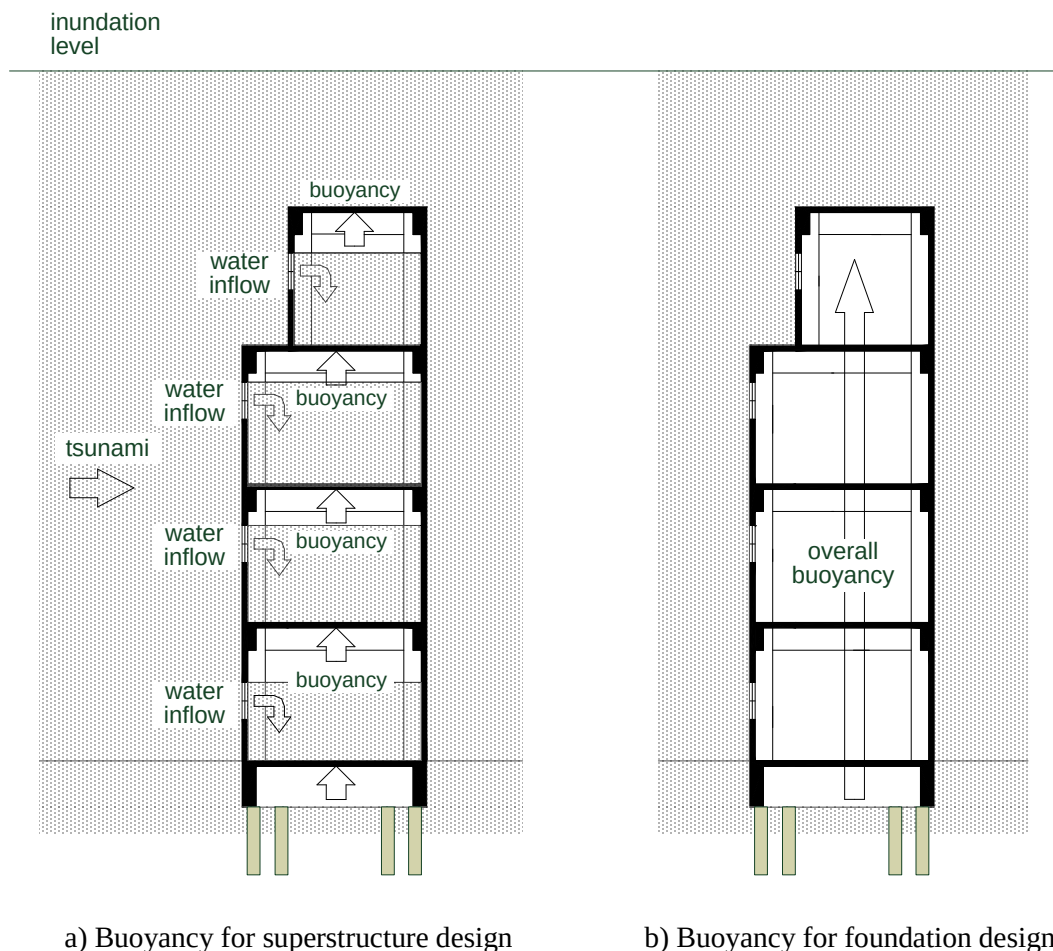


Figure 6. Buoyancy assumptions in the Guidelines 2011. (after NILIM 2012)

- a) In computing lateral strength of structural members for collapse prevention, buoyancy due to water volume displaced by the members and additional buoyancy due to air trapped below the floor system should be considered to assume minimal axial force contributing to the member resistance. (Figure 6 a))
- b) In computing axial forces acting on foundation piles against overturning and friction resistance between foundation and superstructure against lateral movement, potential uplift from the overall buoyancy of the building should be considered to assume maximal uplift forces for the foundation design. (Figure 6 b))

Design of components

Components on the exterior frames are directly exposed to tsunami pressure, and they are categorized either as breakaway or non-breakaway elements. A breakaway component is not part of the structural support of a building and is intended through its design and construction to collapse under a specific tsunami load without causing damage to the building system. Nonstructural components on exterior frames that are directly exposed to tsunami wave pressure are considered breakaway panels and the tsunami loads acting on them can be accordingly reduced if the allowable loading resistance of the components can be given and is lower than tsunami loads. In particular, the contribution of standard residential entry doors, shutter doors, windows on an exterior frame that are expected to fail during the early phase of tsunami attack can be neglected in calculating tsunami loads acting on a structure. Other nonstructural components may be considered breakaway panels and need not to be designed to resist the anticipated tsunami loads but structural components (and therefore the entire structure) supporting them should be designed to effectively resist the loads transferred by such nonstructural components before they fail. Structural components are non-breakaway components and should be properly designed to resist the design tsunami load.

Debris impacts

When tsunamis propagate inland, destructive waves can carry debris such as log, vehicles, boats, vessels, shipping container, etc. creating high impact loads. Various types of tsunami-driven debris impact on structures during tsunami inundation. Several different formulas to compute impact forces are proposed. However, the computed values are in general highly dependent on each proposal. Due to uncertainties involved in the estimate of impact forces associated with waterborne debris, missing column strategy is employed in the Guidelines 2011 to minimize the possibility of progressive collapse of the structural system. Because the missing column strategy has been also employed in the Japanese Standard for Seismic Evaluation of Existing RC Buildings since its publication (JBDPA 1977) and has been widely applied to evaluate buildings throughout Japan, the progressive collapse prevention concept and its criteria are therefore well accepted by building engineers in Japan.

Scour effects

Scour around shallow foundations can lead to failure of the supported structure or structural element. In general, tsunami scour depth is difficult to predict because of the many variables that govern the scour mechanism. The current Guidelines 2011 give consideration to scour, but do not provide guidance for calculating the scour depth. To minimize the undesirable effects of scouring on the structural system, as well as of liquefaction induced differential settlement, deep concrete foundations (pile foundations) should be provided for the tsunami evacuation buildings instead of shallow foundations such as mat foundations, because of the scour potential that can occur during a tsunami. Deep concrete foundations extended well below the anticipated scour depth would be the most appropriate building support method. Concrete buildings better survived the tsunami inundation as observed in the 2011 Great East Japan Earthquake.

STRUCTURAL DIMENSION AND RESISTANCE REQUIRED FOR TSUNAMI EVACUATION BUILDINGS

The Guidelines are applied to roughly estimate the structural dimensions and resistance required for the tsunami evacuation buildings. In this example study, a multistory RC apartment house with 3.5m

height in each story, which has a 12m long wall-frame system in short direction and a frame system with a resistance C_{BX} of 0.3 in terms of base shear coefficient in long direction, respectively, is investigated. Parameters are 1.5, 2.0, and 3.0 for the water depth coefficient a and 5m, 10m, and 15m for the inundation depths h , respectively. Considering different combinations of water depth coefficient a and inundation depth h , the design tsunami load is computed in accordance with the Guidelines 2011, where the example building is assumed to have an opening ratio of 0.3 (i.e., tsunami load reduction is 0.7). Then the required resistance C_{BY} in terms of base shear coefficient in short direction and the building length in long direction, are respectively computed, assuming the building is resistive enough to the overturning and lateral movement. Since building engineers in general are well familiar with seismic design but are not with tsunami design, the results are compared to the conventional seismic design procedure. Table 2 summarizes the results. It should be noted that the minimal number of stories for the evacuation building (or the elevation of tsunami refuge areas) is determined from the following equation considering possible splash-up and one story height for additional allowance for freeboard since the highest splash-up at the front face of a building during tsunami impact was 4 m in the field survey after 2011 event (IIS UTokyo 2011). In Eq. (3), the each story height is assumed 3.5m.

$$N_{\min} = \text{round up } [(h + 4.0(\text{m})) / H] + 1 \quad (3)$$

where,

- $N_{\min}$: number of stories required to the building concerned,
- h : design inundation depth (m),
- H : story height (m) (3.5 m assumed herein).

The required base shear coefficient C_{BY} in short direction is 0.97 at maximum for $h=5\text{m}$ (i.e., 5m deep inundation) and is 0.78 for $h=10\text{m}$ and $a=1.5$, respectively, and the building can be designed in the analogous way of standard seismic design practice under these design loads. However, C_{BY} increases to 1.44 and the building should be 60m long for $a=2.0$, and the strength should be higher than required in the standard seismic design practice. Furthermore, C_{BY} is 2.83 for $a=3.0$, and foundation structures would require significantly high resistance as well as superstructure. The building requires much higher resistance for $h=15\text{m}$, as shown in the table.

Table 2. Required base shear coefficient C_{BY} in short direction and building length in long direction under different combinations of water depth coefficient a and inundation depth h .

(after Fukuyama and Nakano 2013)

	Inundation depth h (required minimal number of stories)		
	5m (4F)	10m (5F)	15m (7F)
<u>$a=3.0$</u> Short direction (12m long) Long direction ($C_{BX}=0.3$)	⊙ $C_{BY}=0.97$ 40m long	△ $C_{BY}=2.83$ 36m long (if $C_{BX}=1.0$ provided)	△ $C_{BY}=4.56$ 54m long (if $C_{BX}=1.0$ provided)
<u>$a=2.0$</u> Short direction (12m long) Long direction ($C_{BX}=0.3$)	⊙ $C_{BY}=0.38$ 15m long	○ $C_{BY}=1.44$ 60m long	△ $C_{BY}=2.42$ 54m long (if $C_{BX}=0.55$ provided)
<u>$a=1.5$</u> Short direction (12m long) Long direction ($C_{BX}=0.3$)	⊙ $C_{BY}=0.30$ 9m long	⊙ $C_{BY}=0.78$ 33m long	○ $C_{BY}=1.36$ 54m long

*Each symbol denotes;

⊙: Standard seismic design practice can apply,

○: Standard seismic design practice can apply but supplemental considerations are needed to increase strength, and

△: Special considerations are needed to provide enough strength to foundation structure and superstructure.

In the case study above, the building is assumed to have appropriate safety for overturning and lateral movement. A recent research (Ohta et al. 2013) proposes a simplified procedure to quickly identify the most critical design condition among superstructure's resistance, overturning and lateral movement. The procedure is applied to existing RC buildings and the results point out that the resistance of overturning or lateral movement is generally less than the superstructure's resistance, and that rational schemes to effectively improve their resistance of overturning and lateral movement need to be developed.

CONCLUSIONS

Structural design requirements for tsunami evacuation buildings newly issued in 2011 as the Interim Guidelines (Guidelines 2011) are presented. The Guidelines 2011 generally follow the basic concept found in the previous BCJ Guidelines 2004 and JCO Guidelines 2005 but revisions are made based on the extensive damage surveys made by the IIS UTokyo-BRI joint team including the author. Although the tsunami pressure formula is simply expressed by the form of equivalent hydrostatic concept, the pressure can be reduced depending on the presence/absence of tsunami energy dissipation structures and the distance of the building from shoreline or riverbank. Technical comments and design considerations are also described to help engineers for practical application. However, there still remain design issues to be examined and described more quantitatively such as effects of tsunami velocity on its pressure acting on structures, those of areal ratio of openings on force reduction, forces acting on columns in open space beneath the elevated building, tsunami debris impacts, etc. Further investigations related to the above issues are still on-going for the next revision of the Guidelines.

REFERENCES

- Asai, T., Nakano, Y., Tateno, T., Fukuyama, H., Fujima, K., Sugano, T., Haga, Y. and Okada, T. (2012). "Tsunami Load Evaluation Based on Field Investigations of the 2011 Great East Japan Earthquake.", *Proceedings of the 15th World Conference on Earthquake Engineering (DVD)*, September 24-28, Lisbon, Portugal.
- Asakura, R., Iwase, K., Ikeya, T., Takao, M., Kaneto, T., Fujii, N. and 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. (in Japanese)
- Fukuyama, H. and Nakano, Y. (2012). "Structural design requirement on the tsunami evacuation buildings.", *structure*, No. 123, JSCA(Japan Structural Consultants Association), 50-53. (in Japanese)
- IIS UTokyo/Institute of Industrial Science, The University of Tokyo (2011). "Interim Report of the Building Standards Improvement Project No. 40, *A Study on Improvement of Building Standards in Tsunami Hazard Areas*.", <http://www.mlit.go.jp/common/000172791.pdf> (interim report) and 000172792.pdf (interim report part2). (in Japanese)
- JBDPA/The Japan Building Disaster Prevention Association (1977, 1990, and 2001(in Japanese) and 2005 (in English)). "Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings."
- JCO/Task Committee under the Japanese Cabinet Office (2005). "Design Guidelines for Tsunami Shelters." (in Japanese), <http://www.bousai.go.jp/oshirase/h17/050610/guideline.pdf>
- MLIT/Ministry of Land, Infrastructure, Transportation and Tourism (2011). "Interim Guidelines on Structural Requirements for Tsunami Evacuation Buildings Considering the Great East Japan Earthquake.", *Annex to the Technical Advice*, MLIT, Housing Bureau, Building Guidance Division, No. 2570, Nov. 17, 2011.
- 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. (in Japanese)
- Nakano, Y. (2008). "Design load evaluation for tsunami shelters based on damage observations after Indian Ocean Tsunami disaster due to the 2004 Sumatra Earthquake.", *Proceedings of The 14th World Conference on Earthquake Engineering (CDROM)*, Paper ID 15-0008, October 12-17, 2008, Beijing, China.

- NILIM/National Institute for land and Infrastructure Management (2012). “Commentary on Structural Requirements for Tsunami Evacuation Buildings.” (in Japanese), http://www.kenchiku-bosai.or.jp/seismic/tsunami_text.html
- Ohta, T., Sakuta, J., Mochizuki, S., Yanagi, T., Choi, H., And Nakano, Y. (2013). “Fundamental Study on Simplified Estimation of Tsunami Resistant Capacity of Reinforced Concrete Buildings (Part1. Feasibility Study on Tsunami Capacity of Existing Buildings and Part2. Verification of Simplified Estimation).”, *Summaries of technical papers of Annual Meeting*, Architectural Institute of Japan, Structures IV, 649-652. (in Japanese)
- Okada, T., Sugano, T., Ishikawa, T., Ogi, T., Takai, S. and Hamabe, T. (2004a). “Structural Design Method of Building to Seismic Sea Wave.”, *No. 1 Preparatory Examination. Building Letter*, The Building Center of Japan. (in Japanese)
- Okada, T., Sugano, T., Ishikawa, T., Ogi, T., Takai, S. and Hamabe, T. (2004b). “Structural Design Method of Building to Seismic Sea Wave, No. 2 Design Method (a Draft).” *Building Letter*, The Building Center of Japan. (in Japanese)
- Okuda, Y. and Sakata, M. (2008). “Simulation of Tsunami on a Building part 2: Effects of Openings of a Building.”, *Summaries of technical papers of Annual Meeting*, Architectural Institute of Japan, B-1, Structures I, 77-78. (in Japanese)

THE ASCE 7 TSUNAMI LOADS AND EFFECTS DESIGN STANDARD FOR THE U.S

Gary Chock, S.E., F.SEI, F.ASCE, D.CE¹

Introduction: The Tsunami Loads and Effects Subcommittee of the ASCE/SEI 7 Standards Committee has developed a new Chapter 6 - Tsunami Loads and Effects for the 2016 edition of the ASCE 7 Standard, *Minimum Design Loads for Buildings and Other Structures*. Chapter 6 provides loads and other requirements for tsunami and its effects. The 2016 edition of the ASCE 7-16 Tsunami Loads and Effects chapter will be applicable initially to the states of Alaska, Washington, Oregon, California, and Hawaii, which are tsunami-prone regions that have probabilistically quantifiable hazards resulting from tsunamigenic earthquakes of subduction mechanism.

The International Building Code (IBC) references design provisions that are given in American Society of Civil Engineers Standard 7. The ASCE 7 Standard becomes part of an enacted building code law through adoption of the model International Building Code by the local authority having jurisdiction (such as a state, county, or city). The IBC would incorporate ASCE 7-16 in 2018. Therefore, it is anticipated that the first national tsunami design provisions of ASCE 7-16 would be utilized as a part of mandatory building codes of U.S. jurisdictions after 2020.

In these five western states, it is recognized by federal, state, and local governments that mitigation of tsunami risk to public safety requires emergency preparedness for evacuation, in addition to structural resilience of critical facilities necessary for immediate response and economic and social recovery. The public safety risk has been only partially mitigated through warning and preparedness of evacuation; there are many areas where complete evacuation cannot be assured. The lesson of recent catastrophic tsunamis is that historical records alone do not provide a sufficient measure of the potential heights of future tsunamis. Engineering design must consider the occurrence of events greater than scenarios in the historical record, based on the underlying seismicity of subduction zones. For U.S. national tsunami design provisions to achieve a consistent reliability standard of structural performance for community resilience, Probabilistic Tsunami Hazard Analysis (PTHA) consistent with source seismicity is performed in addition to consideration of historical event scenarios.

Keywords: tsunami, loads, design, codes, standards, ASCE 7, probabilistic, hazard, analysis, PTHA

ORGANIZATION OF THE TSUNAMI LOADS AND EFFECTS – CHAPTER 6 OF ASCE 7

Chapter 6 of ASCE 7-16 (ASCE, 2016) is organized to provide, in general order, its scope requirements, methods for the determination of design inundation depth and flow velocities at a site, tsunami loads for structural design, and associated criteria for foundations, other systems, and Tsunami Vertical Evacuation Refuge Structures (Table 1 lists the sections in detail.)

¹ Structural Engineer, Hawaii and California, ASCE Structural Engineering Institute Fellow, ASCE Fellow Diplomat, Coastal Engineer, of the Academy of Coastal, Ocean, Port and Navigation Engineers; Chair, ASCE 7 Tsunami Loads and Effects Subcommittee; President, Martin & Chock, Inc., Honolulu, Hawaii / USA; gchock@martinchock.com

Table 1 Section Topics and Subject Matter of ASCE 7-16 Chapter 6 - Tsunami Loads and Effects

Section	Subject Matter
6.1 General Requirements	Scope
6.2 Definitions	Terminology
6.3 Symbols and Notation	
6.4 Tsunami Risk Categories	Risk Categories
6.5 Analysis of Design Inundation Depth and Velocity	Depth and Velocity
6.6 Flow Depths and Velocities Based on Runup	Determination
6.7 Flow Depths and Velocities Based on Site-Specific Probabilistic Tsunami Hazard Analysis	
6.8 Structural Design Procedures for Tsunami Effects	Basis of Design
6.9 Hydrostatic Loads	Fluid Loads
6.10 Hydrodynamic Loads	
6.11 Debris Impact Loads	Debris Loads
6.12 Foundation Design	Foundation
6.13 Structural Countermeasures for Tsunami Loading	Mitigation
6.14 Tsunami Vertical Evacuation Refuge Structures	Evacuation
6.15 Designated Nonstructural Components and Systems	Other Systems
6.16 Non-Building Structures	
6.17 Consensus Standards and Other Referenced Documents	Referenced Standards

TSUNAMI RISK CATEGORIES

The ASCE 7 Standard classifies facilities in accordance with Risk Categories that recognize the importance or criticality of the facility. The tsunami design requirements in the ASCE 7 Standard vary by Risk Category and height. Risk Category III and Risk Category IV structures in the Tsunami Design Zone are subject to the ASCE tsunami provisions. Hydrodynamic and impact forces are required to be increased by an Importance Factor depending on the Tsunami Risk Category.

The provisions do not apply to single-family homes and other light-frame structures, nor do they apply to single-story and low-rise Risk Category II structures. When communities with public awareness are enabled with tsunami warning systems with emergency operations plans for evacuation, low-rise Risk Category II buildings are not intended to be occupied during a tsunami.

Conversely, whenever evacuation from the inundation zone is not possible within the available time, the public will attempt to use taller buildings to escape the tsunami inundation, and they will inherently expect that such taller structures will not collapse during the tsunami. Under such circumstances the availability of taller buildings that are tsunami resistant is a direct benefit to public safety (Fraser, et al., 2012). For these reasons, it is recommended that the building code of a local jurisdiction also make ASCE 7-16 applicable to multistory buildings at least one-story taller than the maximum expected inundation depth within its Tsunami Design Zone. Where it is required by a state or locally adopted building code statute to include tsunami design of Tsunami Risk Category II buildings, a threshold mean height above grade plane (the height perhaps combined with the occupancy classification) should be designated for the criteria of applicability. The threshold height would depend on their community's tsunami hazard, its tsunami response procedures, whole community disaster resilience goals, and zoning regulations. Evacuation egress time from a local community would also be a consideration.

PROBABILISTIC TSUNAMI HAZARD ANALYSIS

For U.S. national tsunami design provisions to achieve a consistent reliability standard of structural performance for community resilience, Probabilistic Tsunami Hazard Analysis (PTHA) consistent with source seismicity is performed in addition to consideration of historical event scenarios. PTHA generates large probabilistic catalogs of tsunami waveforms directly from the source mechanisms in accordance with logic tree probabilities for each possible subduction source mechanism (e.g., slip distribution and extent of rupture) consistent with their estimated plate convergence rates, and propagates these waveforms to the offshore regimes of the coastlines. The PTHA process was refined in accordance with a peer review study sponsored by the National Tsunami Hazard Mitigation Program (California Geological Survey, 2015).

PTHA results are first embodied in the Offshore Tsunami Amplitude maps of the ASCE 7 Tsunami Design Geodatabase. These offshore hazard maps are defined at a bathymetric water depth contour of 100-meter offshore and serve to document the regional probabilistic tsunami hazard. The Maximum Considered Tsunami (MCT) is defined for a 2% probability of being exceeded in a 50-year period, or a 2475-year average return period. The MCT is the design basis event, consisting of the inundation depths and flow velocities at the stages of in-flow and out-flow most critical to the structure.

ANALYSIS OF DESIGN INUNDATION DEPTH AND VELOCITY

There are two procedures for determining the flow depth and velocities at a site: 1) the Energy Grade Line Analysis, which takes the runup elevation and inundation limit indicated on the Tsunami Design Zone map as the target solution point of a hydraulic analysis along the topographic transect from the shoreline to the runup point; the Energy Grade Line determines the variation of inundation depth and associated flow velocity across the inland profile, and 2) a two-dimensional Site-Specific Inundation Analysis that utilizes the Offshore Tsunami Amplitude, the wave period that is considered a conserved property, and other waveform shape parameters as the input to a numerical simulation that includes a high-resolution digital elevation model of nearshore bathymetry and onshore topography. As illustrated in Figure 1, the Offshore Tsunami Wave Amplitude map and associated waveform parameters are calibrated to the 2,475-year hazard level to define the Hazard Consistent Tsunami. The primary purpose of this information is to support Site-Specific Inundation Analysis of Risk Category IV structures where more detailed spatial bathymetry and topography are used with one of the NOAA-benchmarked 2D inundation model software codes to determine the time history of flow depth and velocity parameters for the site.

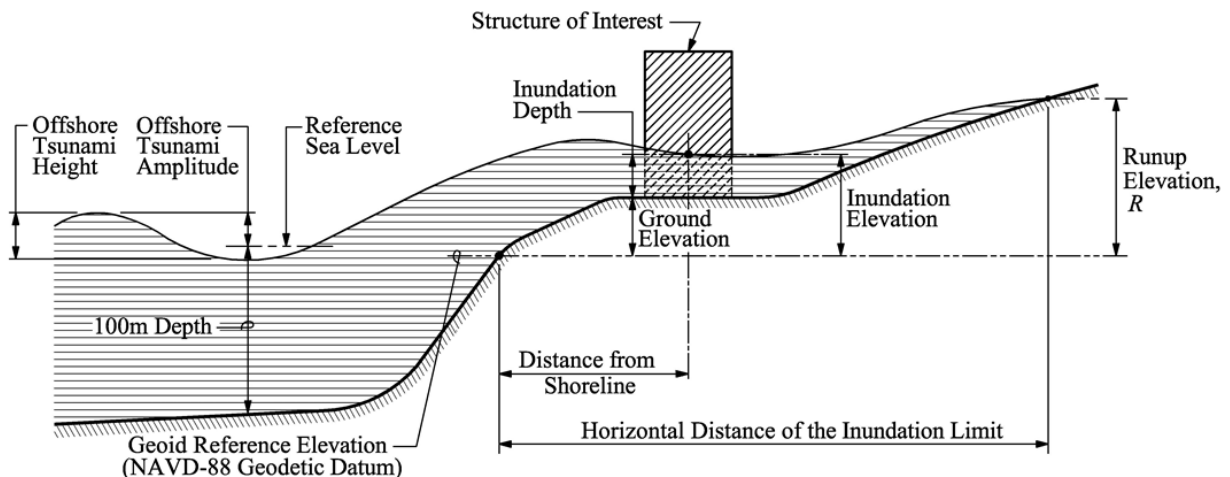


Figure 1 Illustration of Key Definitions along a Flow Transect in a Tsunami Design Zone

The Tsunami Design Zone is the area vulnerable to being flooded or inundated by the Maximum Considered Tsunami. Key parameters for the tsunami basis of design are the Offshore Tsunami Amplitude, Inundation Depth, Runup, and (horizontal) Inundation Limit. Inundation Depth is the depth of tsunami water level with respect to the local grade plane at a site. The Inundation Limit is the maximum horizontal extent of the inundation zone relative to the shoreline at Mean Sea Level. Runup is the elevation above reference datum at the tsunami Inundation Limit. The Runup of the Maximum Considered Tsunami was used by ASCE to develop the Tsunami Design Zone maps using state-of-the-art numerical simulation inundation modeling.

STRUCTURAL DESIGN PROCEDURES FOR TSUNAMI EFFECTS

Building components are simultaneously subjected to internal actions generated by the external loading on the lateral-force-resisting system, together with high intensity momentum pressure forces directly exerted on individual members (Chock, et al. 2013). Three cases of tsunami loading defined by inundation depths and their associated velocities are required to be considered.

1. Inundation depth of the lesser of one-story or the height of the top of the first story windows, when the minimum condition of combined hydrodynamic force with buoyant force shall be evaluated with respect to the depth of water in the interior.
Exception: Load Case 1 need not be applied to Open Structures nor to structures where the soil properties or foundation and structural design prevents detrimental hydrostatic pressurization on the underside of the foundation and lowest structural slab
2. Two-thirds of maximum inundation depth when velocity is near maximum, in each direction
3. Maximum water depth when the flow velocity is assumed to be at one-third maximum, in each direction

Loads on initially enclosed buildings should be calculated assuming a minimum closure ratio of 70% of the pressure exposed surface area of the exterior enclosure; this accounts for accumulated waterborne debris trapped against the side of the structure as well as any internal blockage caused by building contents that cannot easily flow out of the structure. As a practical matter based on observations of buildings subjected to destructive tsunami, “breakaway” walls cannot be relied upon to relieve structural loading, primarily due to the copious amount of external debris. Also, studs and girts may be capable of entrapping contents within a building, thus generating hydrodynamic drag forces on the internal debris that in turn transfer those loads to the structure.

Load Combinations

Structures and structural components shall be designed for the Tsunami Forces and Effects, F_{TSU} , the principal load effect, for incoming and receding directions as specified in the load combinations of Eq. 1 and Eq. 2:

$$0.9D + F_{TSU} + H_{TSU} \quad (1)$$

$$1.2D + F_{TSU} + 0.5L + 0.2S + H_{TSU} \quad (2)$$

where for the definition of load combinations,

D = dead load

L = live load

S = snow load

F_{TSU} = tsunami load effect for incoming and receding directions of flow

H_{TSU} = load due to tsunami-induced lateral foundation pressures developed under submerged conditions. Where the net effect of H_{TSU} counteracts the principal load effect, the load factor for H_{TSU} shall be 0.9.

Structural Loads and Effects

The following tsunami effects are considered for the structural design of buildings and structures:

- hydrostatic forces, buoyant forces, and additional fluid gravity loads from retained water;
- hydrodynamic forces and hydrodynamic uplift forces;
- debris impact forces that depend on the sources of debris in the proximity of the site
- foundation scour and pore pressure softening effects on the soil

Hydrostatic Loads

Reduced net self-weight due to buoyancy should be evaluated for all inundated structural and non-structural elements of the building. Buoyancy shall also include the effect of air trapped below floors, including integral structural slabs, and in enclosed spaces where the walls are not designed to break away. All windows, except those designed for large missile wind-borne debris impact or blast loading, shall be permitted to be considered openings when the inundation depth reaches the top of the windows or the expected strength of the glazing, whichever is less.

Hydrodynamic Loads

Hydrodynamic loads develop when fluid flows around objects in the flow field. The effect of flow laden with small debris is accounted for with an increased effective fluid density. The building lateral framing system should be designed to resist the overall drag force developed either by in-coming or out-going flow surge. Likewise, lateral hydrodynamic pressure load should be applied on the projected area of all structural elements and enclosure component assemblages below the inundation flow depth.

Debris Impact Hazard Regions

Tsunamis can transport a large volume of debris. Virtually anything in the flow path that becomes buoyant in the inundation depth or that cannot withstand the hydraulic forces will become debris. Common examples are trees, wooden utility poles, cars, and wood frame houses and portions thereof. Waterborne debris impact is applied to any perimeter structural element of the gravity-load-carrying system within the inundation depth at the site. A dynamic impact of a 1,000-lb. log or similar object traveling at the maximum flow velocity, a floating passenger vehicle force of 30,000 lbs., and an 8,000-lb. impact force by submerged tumbling boulder (or concrete mass debris) should be assumed to impact perimeter vertical structural elements of the gravity-load-carrying system.

Other debris impact forces are applicable depending on the location of the structure and potential special debris such as shipping containers in the surrounding area that would be expected to reach the site during the tsunami (Riggs, et al, 2014). To determine the extent of the debris impact hazard region around a port, harbor, or shipping container yard, an empirical approach (Riggs, et al., 2015) is based on the amount of available large floatable debris and the flow depth in the vicinity, along with other significant structures that may be between the origin of debris and the structure being designed. In a bi-directional sectorized region, there is a greater probability of receiving a colliding impact (illustrated in Figure 2).

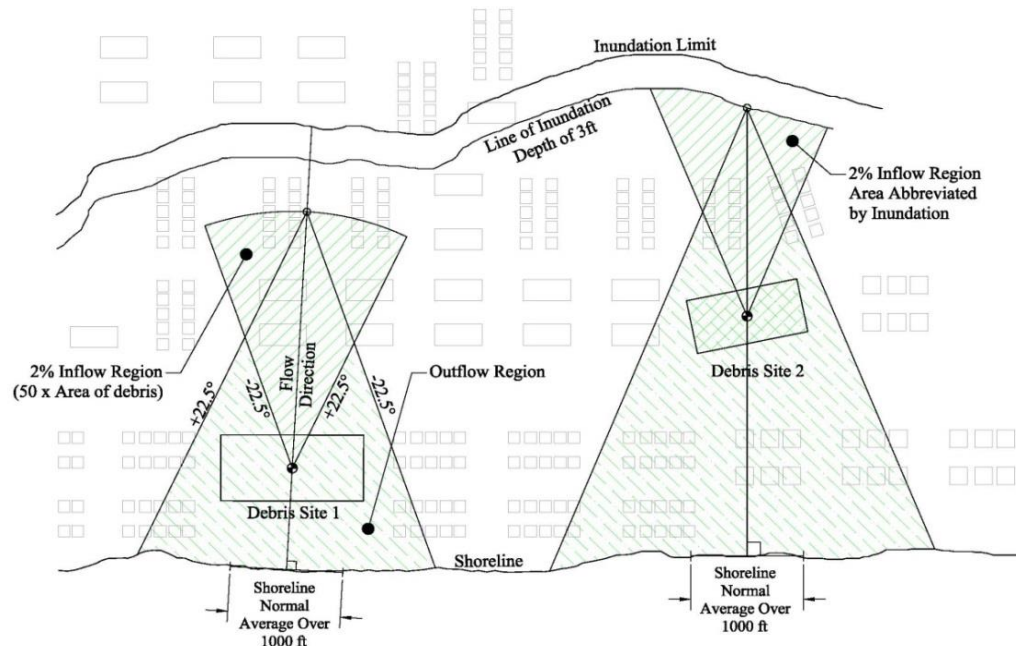


Figure 2. Illustration of Determination of Floating Debris Impact Hazard Region

For buildings and other structures within the debris impact hazard region of a shipping port, a shipping container impact on perimeter vertical structural elements of the gravity-load-carrying system should be assumed to occur. Adjacent to piers and wharves, an extraordinary mass impact (such as a large ship) on perimeter vertical structural elements of the gravity-load-carrying system of Risk Category III and IV buildings and structures should be evaluated. The most severe effect of impact loads within the inundation depth should be applied to the perimeter gravity-load-carrying structural elements located on the principal structural axes perpendicular to the range of inflow or outflow directions.

Foundation Design

Design of structure foundations and tsunami barriers should consider changes in the site surface and in-situ soil properties during the design tsunami. Tsunami flow around structures can cause local scour around foundation elements, and sustained flow can also result in general erosion at a site. Tsunami inflow and outflow cycles are specified to include load reversal as well as scour effects that may occur due to an initial wave prior to a subsequent wave loading. Loading should consider a minimum of two tsunami in-flow and out-flow cycles, the second of which should be at the maximum design level. This is required because the condition of the building and its foundation is altered in each load reversal and through each tsunami inflow and out-flow cycle. Deep foundations should be designed to resist vertical and lateral tsunami loads during and after combined general site erosion and scour. Foundation embedment depth and the capacity of the exposed piles to resist structural loads, should take into account the cumulative effects of general erosion due to flooding and pore-pressure softening, as well as local scour. Figure 3 presents a schematic representation of the applicable loading on a foundation element for the design condition after local scour and general erosion have occurred and pore pressure and seepage effects are present. This diagram may also apply to an integrated foundation system (such as a mat) as a whole.

For nonlinear materials, such as soils, a limit state is assumed to exist along some failure surface and the resultant actions from an equilibrium analysis are compared to the reduced nominal strength for that material. Hence, this approach is commonly called limit-equilibrium analysis. To ensure that the assumed failure does not occur, a resistance factor is applied to the material nominal strength. Typical failure mechanisms evaluated in foundation stability analyses are:

- Lateral sliding (due to tsunami forces with the added effects of any unbalanced lateral soil pressures caused by local scour on one side)
- Uplift or floatation
- Slope stability analysis (due to saturation and pore pressure softening effects of inundation)
- Bearing Capacity (where soil strength properties may be affected by sustained pore pressures, however, lateral sliding failures typically occur before actual bearing stress failures)

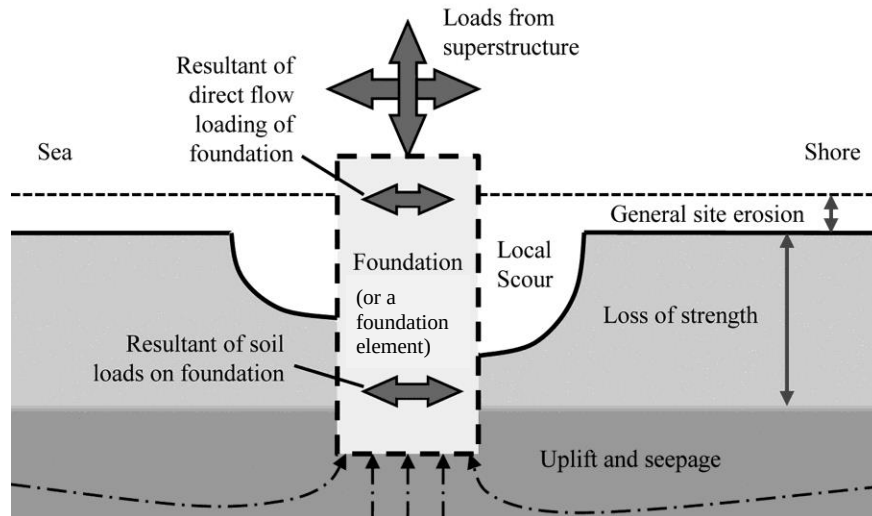


Figure 3. Schematic of tsunami loading condition for a foundation or foundation element.

Foundation and Structural Countermeasures for Tsunami Loads and Effects

The potential extreme magnitude or severity of tsunami loading warrants use of robust or redundant structural countermeasures including open structures, and use of tsunami mitigation barriers capable or resisting tsunami loading and scour, located in positions to protect important buildings or communities.

Vertical Evacuation Refuge Structures

Vertical Evacuation Refuge Structures are a special classification of buildings and structures within the tsunami evacuation zone designated as a means of alternative evacuation in communities where sufficiently high ground does not exist, or where the time available after the tsunami warning is not deemed to be adequate for complete evacuation prior to tsunami arrival. A particularly important consideration is the elevation and height of the refuge, since it must provide structural life safety for the occupants within a portion of the refuge that is not inundated. Therefore, additional conservatism is necessary in the estimation of inundation height. The minimum elevation for a tsunami refuge area is, therefore, the Maximum Considered Tsunami Runup elevation anticipated at the site, increased by 30%, plus 10 feet (3 meters), as illustrated in Figure 4.

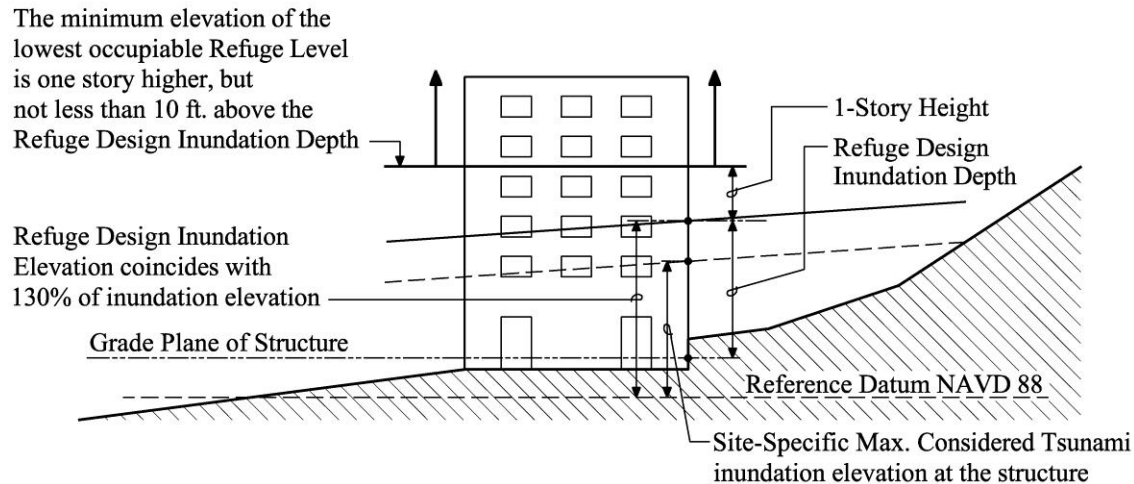


Figure 4. Minimum Refuge Elevation and Design Inundation

Designated Nonstructural Components and Systems

Designated nonstructural components and systems require special attention since they are needed to continue to perform their function following both earthquake and tsunami events. From a tsunami perspective, there are two approaches that can be used to better assure that designated nonstructural components will perform as needed. One approach is to locate the components in the structure of concern above the Maximum Considered Tsunami inundation elevation. The other approach is to protect the components from inundation effects.

Non-Building Structures

From a tsunami perspective, there are four or more approaches that can be used to design non-building structures to resist tsunami effects. The first is to design the structure and its foundation to resist the effects of tsunami forces directly. The second is to locate the nonbuilding structure safely above the Maximum Considered Tsunami maximum inundation elevation. The third approach is to protect the components from inundation effects. A fourth approach is to design a protective barrier to mitigate the flow depth to a level sustainable by the structure (rather than keeping the structure entirely dry.)

SUMMARY

A method of Probabilistic Tsunami Hazard Analysis has been established for determining the Maximum Considered Tsunami. The tsunami design provisions utilize procedures for tsunami inundation analysis based on using design map values of Offshore Tsunami Amplitude or the Runup and Inundation Limit from the Tsunami Design Zone Map. Structural loading and analysis techniques for evaluating building performance are included. The new ASCE 7 provisions for Tsunami Loads and Effects enables a set of analysis and design methodologies that are consistent with probabilistic hazard analysis, tsunami physics and hydraulic engineering principles.

REFERENCES

- American Society of Civil Engineers, Structural Engineering Institute (2016). *Minimum Design Loads for Buildings and Other Structures*, ASCE/SEI 7-16 (in preparation for publication).
- California Geological Survey (2015). *Evaluation and Application of Probabilistic Tsunami Hazard Analysis in California*, Special Report 237, Sacramento.

- Chock, G., Robertson, I., Kriebel, D., Francis, M., and Nistor, I. (2013). *Tohoku Japan Tsunami of March 11, 2011 – Performance of Structures under Tsunami Loads*, American Society of Civil Engineers, Structural Engineering Institute, pp. 350.
- Fraser, S., Leonard, G.S., Matsuo, I., Murakami, H. (2012). *Tsunami evacuation: Lessons from the Great East Japan earthquake and tsunami of March 11th 2011*, GNS Science Report 2012/17, 89 pages.
- Riggs, H. R., Cox, D. T., Naito, C. J., Kobayashi, M. H., Piran Aghl, P., Ko, H. T.-S., and Khowitar, E. (2014). “Experimental and analytical study of water-driven debris impact forces on structures,” *Journal of Offshore Mechanics and Arctic Engineering*, doi:10.1115/1.4028338, OMAE-13-1042.
- Riggs, H. R., Naito, C. J., Cercone, C., and Wei, Y. (2015) “ASCE 7 Structural Load Provisions for Impact by Tsunami-driven Shipping Containers, Coastal Structures and Solutions to Coastal Disasters Joint Conference, COPRI, September 9-11, 2015.

Evaluation of Tsunami Force Acting on Bridge Girders

**Kyuichi Maruyama¹, Yasushi Tanaka², Kenji Kosa³, Akira Hosoda⁴,
Taro Arikawa⁵, Norimi Mizutani⁶, and Tomoaki Nakamura⁷**

Synopsis: This paper deals with what tsunami force acted on bridge girders by Great East Japan Earthquake broken out in March 11, 2011. First of all, a lot of efforts were conducted to collect almost all data of bridge girders in the inundation area. Satellite images in internet websites proved very effective to make a quick survey of any bridge in the inundation area. Detailed data on the bridges were obtained from authorities for use in damage analysis. The method proposed by Prof. Kosa was introduced to see whether bridges were washed away or not by tsunami. The resistance of each bridge is expressed as a function of the self-weight of bridge girder, and the tsunami force is defined by hydro-dynamic equation. More than two hundred bridges, including both washed away bridges and survived ones, were analyzed. Motion pictures taken by digital camera during the attack of tsunami were examined to evaluate the velocity and height of tsunami. As far as bridges in a local area were taken into account, the method was good to see whether bridges were survived or not. Field survey of the bridges washed away indicated that the failures would have been by tsunami uplift force. Experimental investigation was done using a water channel to examine what forces acted on bridge girders and to develop a proper simulation model.

Keywords: Tsunami, bridge girder, satellite image, damage analysis, simulation

INTRODUCTION

Great East Japan Earthquake in March 11, 2011 caused the greatest tsunami we have ever had in one thousand years in Japan [1]. Compared to the damage caused by the earthquake itself, the tsunami damage was much more serious impact to modern society in the eastern Japan area. It was a shocking scene that a lot of bridges near the shore were washed away while very few bridges inland suffered from the earthquake. Since the Great Hanshin Earthquake in 1995, bridge engineers and researchers have been working hard on how to improve the seismic design of bridge structures. The damage of bridge by tsunami was completely out of our concerns because we have never had such damage since modern bridge technology has been introduced in one hundred years. Right after the earthquake, JSCE has set up a research committee on the tsunami force acting to bridges, consisting of members from various fields such as bridge engineering, concrete engineering, structural engineering and coastal engineering. The authors have been involved in the committee as directive members.

DAMAGE OF BRIDGES BY TSUNAMI

At the first stage any data of bridges which located in the inundation area from Aomori prefecture in the north to Chiba prefecture in the south were collected by both internet media and on site survey [2]. Fig.1 shows satellite images taken before and after the earthquake.

¹ Professor emeritus at Nagaoka University of Technology, Nagaoka, Niigata, Japan

² Associate professor at the University of Tokyo, Tokyo, Japan

³ Professor at Kyushu Institute of Technology, Tobata, Fukuoka, Japan

⁴ Associate Professor at Yokohama National University, Yokohama, Kanagawa, Japan

⁵ Senior Researcher at Port and Airport Research Institute, Yokosuka, Kanagawa, Japan

⁶ Professor at Nagoya University, Nagoya, Aichi, Japan

⁷ Designated Lecturer at Nagoya University, Nagoya, Aichi, Japan

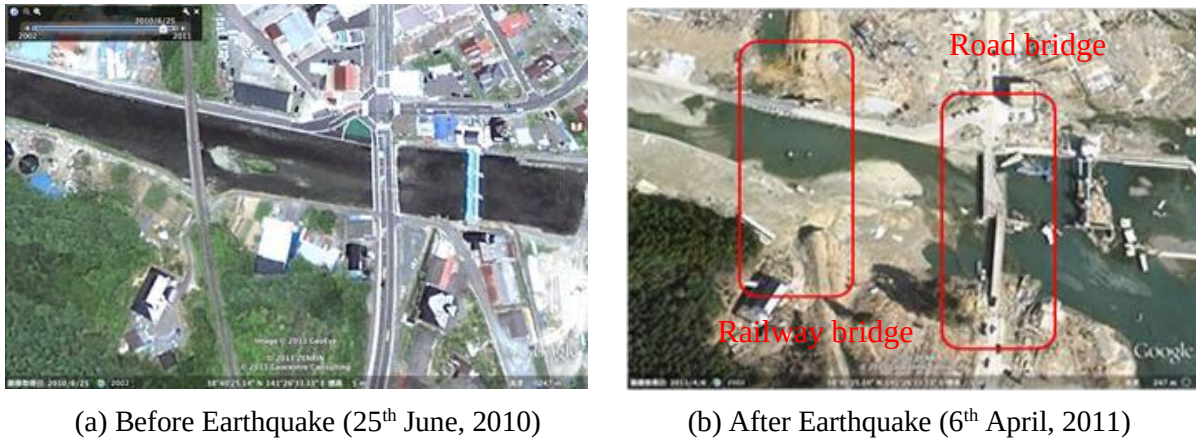


Figure 1. Satellite images.

Field survey was conducted several times to confirm the details of bridges. Finally, the statistic data are summarized as indicated in Table 1. The total number of bridges within the inundation area was 1,793 and the number of washed away bridges was 252, including 28 of railway bridges and 40 of major road bridges.

Table 1. Number of bridges in the area of inundation.

	Iwate	Miyagi	Fukushima	Ibaragi	Chiba	Total
Washed away or Moved	78	102	68	2	2	252
Survived	427	692	384	30	-	1533
Unclear	0	0	8	0	0	8
Total	505	794	460	32	2	1793

In order to analyze the damage of bridges, the data of each bridge, such as bridge type (concrete, steel, composite), dimension and structural details were collected from authorities who take care of bridges. Currently, the data of more than 400 bridges are collected and the database is now processed. Using the database, some trials have started to categorize bridges by their types and dimensions to see what a key is for resisting tsunami. Fig.2 shows the number of bridges with or without suffering from washed away in comparison with categorized items, such as type of bridge and bridge length. In the figure, only the survived bridges located near the washed away bridges are selected for comparison. The figure may not give us a clear tendency on what influences the damage of bridge. This is because no consideration is taken for the velocity and the height of tsunami at the location of bridge.

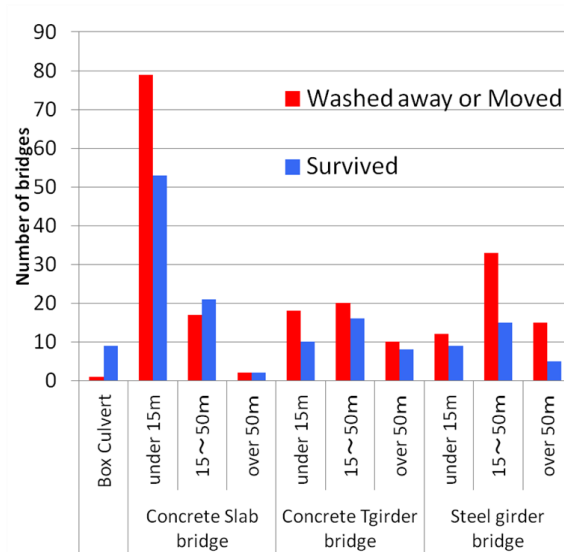


Figure 2: Damage analysis of bridge girders

EVALUATION OF GIEDER RESISTANCE AGAINST TSUNAMI

Kosa, one of the authors, has already started the investigation on the tsunami force acting on bridge girders after surveying the damage of bridges due to the tsunami of Sumatra Earthquake in 2004[3,6,7]. The following discussions are based on his method. Tsunami force acting to a bridge girder can be treated as a hydro dynamic force indicated by Eq.1, while the resistance of girder is mainly due to self weight computed by Eq.3 or Eq.4. The difference between Eq.3 and Eq.4 lies in the existence of buoyancy judging from the comparison of girder height with tsunami height. Eq.3 should be used when a bridge girder is not submerged at the attack of tsunami, while Eq.4 is for the case of bride girder being submerged and having buoyancy at tsunami.

$$F = \frac{1}{2} \rho_w C_d v^2 A_h \quad (1)$$

$$C_d = \begin{cases} 2.1 - 0.1 \cdot (B/D) & (1 < B/D < 8) \\ 1.3 & (8 \leq B/D) \end{cases} \quad (2)$$

$$S = \mu W \quad (3)$$

$$S = \mu(W - U) \quad (4)$$

Where, ρ_w is density of water (1030kg/m³); C_d is the drag coefficient with its value decided based on reference (Japan Road Association [4]), B and D are width and depth of girder; v is tsunami velocity (6.0m/s is used based on surveys of the Tohoku area); A_h is the projected pressure area of the girder in the horizontal direction; μ is friction coefficient (0.6, based on research by Rabbat et al. [5]); W is the girder's self weight (kN), and U is buoyancy (kN) as computed by Eq. 5.

$$U = \rho_w g V \quad (5)$$

Where, V is the displaced volume of the bridge girder within the tsunami inundation depth (m³).

The ratio of girder resistance (S) to tsunami force (F) is an index to judge whether a bridge girder is washed away or not by tsunami. The indicator β is introduced in this paper as shown in Eq. 6.

$$\beta = \frac{S}{F} \quad (6)$$

When β is smaller than 1.0, a bridge girder has a great possibility of being washed away by a given magnitude of tsunami. Although the velocity of tsunami is most important in this study, observe records of velocity of tsunami are very few at the site of bridge. In this study, some recorded videos throughout the Tohoku area were used for determination of velocity. The average value of velocity is taken as 6.0m/s based on the research by Fu et al. [6] and this value is used for analysis.

For evaluation of Eq.6 with comparison of the actual behavior of bridge girder, the observed condition of girder is defined as shown in Table 2. Three levels are introduced; Rank A (completely washed away) to Rank C (slightly damaged).

Table 2. Definition of damage of bridge girder.

Damage level	Observed condition of superstructure
Rank A	Washed away
Rank B	Moved but not fallen from abutment
Rank C	Slightly damaged

Fig.3 illustrates the computed β values for 37 different girders. In general, girders having the β value being larger than 1.5 are categorized in Rank C while girders with β being smaller than 0.6 suffered the damage in Rank A.

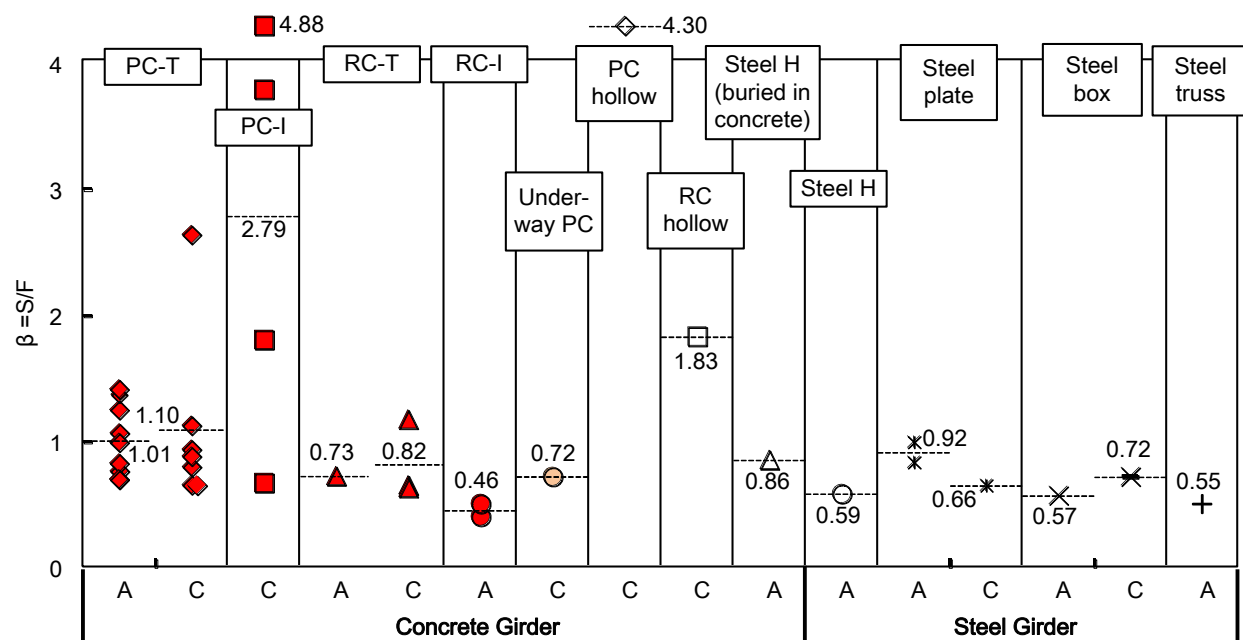


Figure 3: Classification of β by girder type

There observes some girders survived (in Rank C) having β values being less than 1.0. In the previous research works done by Kosa et al. [3]; Zheng et al. [7], obvious differences are recognized in the value of

β for Rank A and Rank C. A variety of types of girders were examined and a general tendency was drowned that those girders having relatively low height and wider width of cross section exhibit higher resistance against tsunami. In addition, the height of pier should play an important role because the velocity of tsunami is a function of tsunami height. In this discussion the tsunami velocity was assumed as 6.0 m/s. Although further development is necessary, the β value defined here could be effective to evaluate how much safe a bridge girder is against tsunami.

CHARACTERISTICS OF TSUNAMI IN THE CITY OF SHIZUGAWA

Japan Meteorological Agent keeps observation of earthquakes and tsunamis constantly at certain points in Japan. The equipments to measure tsunami height and velocity, however, are set in the sea, not inland. The agent does not have any records of tsunami velocity in residential areas. Fortunately, some citizens have taken motion pictures of tsunami in the city by digital cameras.

Based on the motion pictures taken by citizens, the characteristics of tsunami in the city are able to be analyzed. Fig.4 shows a picture of Hachiman Bridge in Shizugawa area in Minamisanriku-Chou, Miyagi prefecture cut out from the recorded motion pictures at 7.05 second. A camera was focused on the bridge named Hachiman Bridge. Fig.5 shows the side view of the bridge. In advance of analysis of picture, a key reference height is estimated as follows; (a) the height from the top of guardrail to static water level was 5.7 m, (b) the height from the top of building-(a) to the top of guardrail was 5.9 m, (c) the height from the top of guardrail to the window of building-(a) was about 2.0 m.

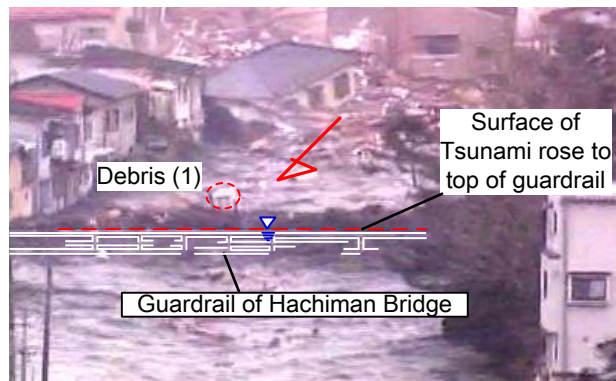


Figure 4. Shot of tsunami wave at Hachiman bridge.

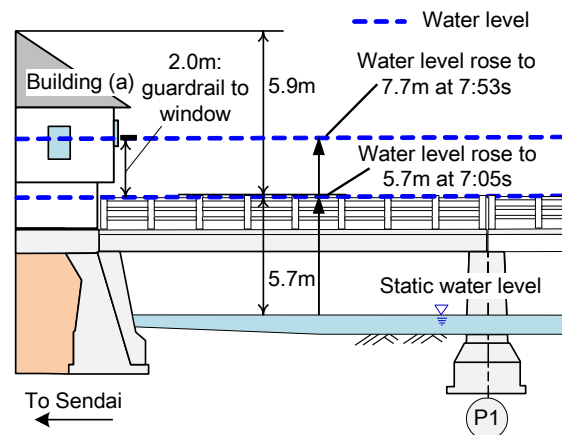


Figure 5. Side view of Hachiman bridge.

The analysis of tsunami is done in such a way that the surface of tsunami reached the top of guardrail at 7:05 second of motion picture, meaning that the tsunami height was 5.7 m. The surface of tsunami reached the window of a house at 7:53 second, meaning that the tsunami height was 7.7 m. Comparing these two sets of data, the rising speed of tsunami can be determined. Motion picture analysis tells that the tsunami height rose with the speed of 3.19 m/min at a time of 4 minutes after the hit of tsunami on Hachiman bridge, and that the tsunami ran up to the maximum 14.8 m high at 7 min. after the hit, at a rising speed of 1.1 m/min. The average rising speed of tsunami height is estimated to be 2.24 m/min. It was relatively low speed, inferring that the wave shape was not the bore type.

There observed much debris flowing in the picture. These are good references to measure the velocity of tsunami. For example, take debris (1) and debris (2) in the motion picture and trace them with a certain

time interval shown in the picture. Accurate distance of debris move can be confirmed by both internet media and on site survey.

By the above mentioned process, the wave height and velocity of tsunami can be estimated. Based on the analyzed results the relationship between tsunami height and tsunami velocity is obtained and shown in Fig.6. In the figure the assumption that the velocity should be 0 when the tsunami height is 0 is added. When the tsunami began to fall back, the velocity was treated as 0 as well. Before the tsunami reached about 2m, the tsunami velocity rose to about 6 m/s.

The velocity of tsunami increased continually before the tsunami wave hit the bridge girder. The maximum velocity is estimated as 7.02m/s just before the tsunami wave reached the bridge girder. When the tsunami wave ran up to about 6m high, the bridge girder began to be inundated and the tsunami velocity got to decrease to about 4.86m/s. After the tsunami height reached about 12m, the velocity decreased to 0 gradually.

Fig.6 tells two things. Firstly, the tsunami velocity continuously increases up to the maximum before the tsunami begins to overflow the bridge girder. Secondly, the tsunami velocity decreased suddenly when the bridge girder is inundated. In addition, Hachiman Bridge has a local geographical feature showing that the level of girder is the same as the level of embankment. When the tsunami height rises up to the level of girder, the tsunami would overflow to the land area and that results in the reduction of tsunami energy and velocity.

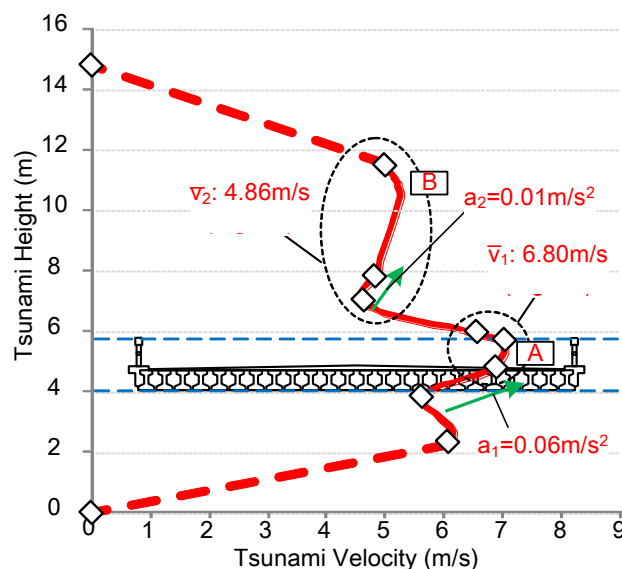


Figure 6. Time-dependent Variation of Tsunami Height.

EXPERIMENTAL EXAMINATION AND SIMULATION

In the previous section the β value is introduced to consider the resistance of bridge girder against tsunami. The β value counts only the horizontal force by tsunami. Field survey, however, indicates the existence of up-lift force due to tsunami. Some bridge girders were washed away without any damage of anti-fallen device provided at the head of piers. Necessary modification for the β value is to introduce the influence

of up-lift force. Experimental works were conducted to examine the mechanism of production of up-lift force on bridge girders during tsunami.

Using relatively large water channel which enable to realize 1/10 scale model of bridge girder, the influence of tsunami velocity, water depth as well as existence of solitary wave as shown in Figs.7 and 8 were examined in terms of up-lift force.

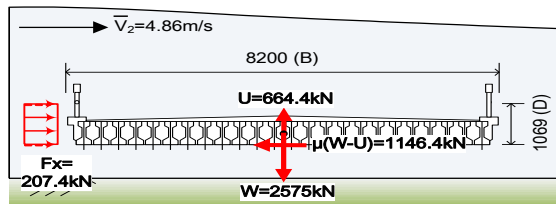


Figure 7. Uniform flow.

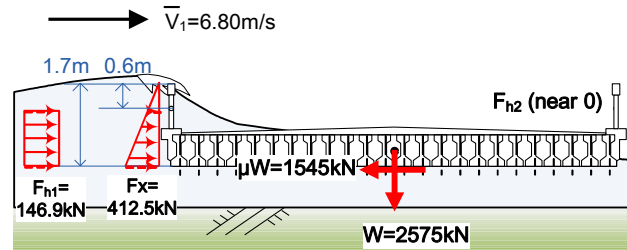


Figure 8. Solitary wave.

The size of water channel was 10 m of width, 4.6m of depth and 50m of length. Two pumps were provided to supply water of 284m³/min. The model scale is 1/10, and on the girders, wave pressure gauges and pour water pressure gauges were installed at 40 locations on the front surface, back surface, top surface, and bottom surface of the girders.

Table 3 shows parameters in this study and Fig.9 shows a schematic view of water depth to a girder. The height of model girder is 1.33m. In the case of uniform flow, the height of water flow increases by 0.10m incrementally. In some cases the velocity of flow is varied by change of amount of water supply. The listed velocity in table 3 was measured ones since flow velocity was not accurately controlled. In the case of solitary wave, basic wave height is 0.20m and the wave height increases by 0.10m incrementally. The initial water depth varies from 1.22 to 1.43. The flow velocity was 1.0 – 1.5 m/s.

Table 3. Experimental parameters.

Uniform flow		Solitary wave	
Initial water depth [m]	Velocity [m/s]	Initial water depth [m]	Wave height[m]
1.33	1.6~1.8	1.22	0.20~0.40
1.43	1.6~1.8	1.33	0.20~0.50
1.53	1.4~2.0	1.43	0.20~0.50
1.63	0.7~1.7		
1.73	1.0~1.5		

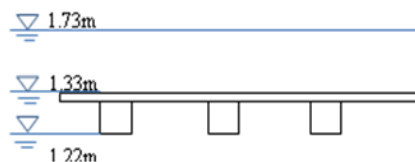


Figure 9. Level of water depth to a girder

Figs. 10 and 11 show pictures of girder in the experiment. In the case of uniform flow (Fig.10) a girder did not move at any water depth or at any water velocity. The similarity rule of experiment indicates that the maximum water velocity in the experiment represents more than 10m/s in the real situation. On the