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PROCEEDINGS OF THE 44TH JOINT MEETING OF U.S.-JAPAN PANEL ON WIND AND SEISMIC EFFECTS UJNR

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PROCEEDINGS OF THE 44TH JOINT MEETING OF U.S.-JAPAN PANEL ON WIND AND SEISMIC EFFECTS UJNR

Edited by

Keiichi Tamura, Secretary-General
Japan-side Panel on Wind and Seismic Effects

Synopsis
This publication contains the proceedings of the 44th Joint Meeting of the U.S.-Japan Panel on Wind and Seismic Effects, UJNR. The meeting was held at the National Institute of Standards and Technology, Gaithersburg, MD, U.S. during February 20-21, 2013. This technical event featured lessons learned from recent disasters, including the 2011 Great East Japan Earthquake.
Keywords: *wind engineering, earthquake engineering, sustainable design, natural disaster prevention, 2011 Great East Japan Earthquake, UJNR*

PREFACE

This publication contains the proceedings of the 44th Joint Meeting of the U.S.-Japan Panel on Wind and Seismic Effects, UJNR. The meeting was held at the National Institute of Standards and Technology, Gaithersburg, MD, U.S. during February 20-21, 2013. The proceedings include the program, list of members, task committee reports, and technical papers submitted to the Joint Panel Meeting.

BACKGROUND

Responding to the need for improving engineering and scientific practices through exchange of technical data and information, research personnel and research equipments, the United States and Japan created the U.S.-Japan Cooperative Science Program in 1961. The U.S.-Japan Cooperative Program in Natural Resources (UJNR) was created in January 1964. The objective of UJNR is to exchange information on research results and scientists and engineers in natural resources of benefit of both countries. UJNR is of 18 Panels each responsible for specific technical subjects.

The Panel on Wind and Seismic Effects was established in 1969. Twenty-one U.S. and seven Japanese agencies currently participate to develop and exchange technologies aimed at reducing damage from high winds, earthquakes, storm surge, and tsunamis. This work is produced through collaboration between U.S. and Japanese member researchers working in five Task Committees. Each Task Committee focuses on specific technical issues, e.g., buildings and infrastructure systems. The Panel provides the vehicle to exchange technical date and information on design and construction of civil engineering infrastructures, buildings, and to exchange high wind and seismic measurement records. Annual Joint Panel Meetings alternate in the U.S. and Japan. These technical meetings including technical site visits provide the forum to discuss on-going research and research results.

The National Institute of Standards and Technology (NIST) provides the U.S.-side chairman and secretary-general. The Public Works Research Institute (PWRI) provides the Japan-side chairman and secretary-general.

These annual Joint Panel Meetings provide the mechanism for interaction with U.S. and Japanese researchers in wind and earthquake engineering which provides opportunities to gain valuable information and to engage in cooperative research. Through these opportunities the Panel member organizations have realized important advances in building and structure technology.

The Panel provides the vehicle to exchange technical data and information on design and construction of civil engineering lifelines, buildings, waterfront, and coastal structures. The data produced by the Panel influence on-going structural engineering research and contribute to the revision and creation of U.S. building codes and standards. Examples of Panel benefits include:

- Created and exchanged digitized earthquake records used as the basis of design and research for Japan and the U.S.
- Produced full-scale test data that advanced seismic design standards for buildings.
- Translated into English a Port and Harbour Research Institute handbook on *Liquefied Remediation of Reclaimed Land*, A. A. Balkema, The Netherlands, publisher that provided general guidance for the US design profession on remediation of liquefiable soils.
- In collaboration with Japan's Geotechnical Society translated into English a report from the Port and Harbour Research Institute, *Remedial Measures Against Soil Liquefaction: From Investigation and Design to Implementation*, A. A. Balkema, The Netherlands, 1998, publisher that served as background and guidance for the Corps of Engineers in performing dam remediation at Clemson University.
- Developed a protocol for testing bridge columns subjected to earthquake loads that facilitated the exchange of experimental data between both countries. The protocol serves as a basis for FHWA's development of new seismic design criteria for bridge columns.
- Facilitated an USACE Team to Kobe within days after the Kobe Earthquake that allowed access to data and information through performing post disaster investigations. This investigation would not have been possible without the Panel's endorsement.
- Performed joint post disaster investigations whose findings influenced revisions to and development of new seismic design and rehabilitation criteria in the US.
- Accessed a large US and Japan database that helped develop an USACE Guidance Criteria in Geotechnical Engineering.
- Provided access to data to help characterize gravely soils in determining the seismic instability of gravely soils for dams and were used to improve USACE construction criteria.
- Through a Japan Guest Researcher from the Port and Harbour Research Institute working at ERDC/WES, Vicksburg geotechnical research findings were transferred into USACE documents.
- Created a database comparing Japanese and US standard penetration tests to improve prediction of soil liquefaction.
- Influenced the creation of a NIST base isolation research program using data from translated Japan reports into English on base isolation systems.
- Increased awareness of wind engineering practice, problems, and breakthroughs in Japan and the U.S.
- Improved cross-discipline research among wind engineers/meteorologists/sociologists in each country.
- Increased the dissemination of latest research findings in wind engineering, especially post-storm events (typhoons/hurricanes) to each country.
- Stimulated interest to create Joint quick-response storm survey teams with interdisciplinary research thrusts to examine storm damage in both countries.
- Developed field test data for use in aerodynamic retrofit of bridge structures.
- Produced data that advanced retrofit techniques for bridge structures.
- Advanced technology for repairing and strengthening reinforced concrete, steel, and masonry structures, improved in-situ measurement methods for soil liquefaction and stability under seismic loads.
- Created database on storm surge and tsunamis and verified mathematical models of tsunami and storm surge warning systems.

- Established a library resource of current research on wind and earthquake engineering and on storm surge and tsunamis.
- Exchanged more than 250 guest researchers between Japan and the US that has resulted in advancing their respective organizations mission research, advanced the state-of-technology, and provided career growth opportunities for these guest researchers.
- Performed joint research in more than 10 collaborative research projects that resulted in new US seismic design criteria for buildings and bridges.
- Published proceedings of Panel meetings, Task Committee Workshops, and special publications such as List of Panel Publications, translated two-volume series on earthquake resistant construction using base isolation systems, newsletter, website of Panel activities, and more.
- Gained better knowledge of both countries research, design and construction capabilities from in-depth visits to host country's laboratories and building and public works projects. Results of such visits contribute to creation of new Task Committees, agendas for Joint Panel meetings and task committee workshops, special visits of US-Japan researchers, and joint collaborative research.

The Panel's efforts are exemplary of effective joint research and of technology delivery between researchers in the U.S. and Japan. Since its creation, about 2000 papers were presented in 40 Joint Panel Meetings and Task Committee Workshops and over 250 guest researchers were exchanged. The Panel provides important information about the U.S. and Japan's civil engineering thrusts which influence both countries' research and provide the basis for improvements in building and structure codes and standards.

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AGENDA OF JOINT MEETING

JOINT MEETING

February 19 (Tuesday)

Arrival of Japan-side Delegation
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February 20 (Wednesday)

8:30 **Opening Ceremonies**
Call to order by Marc Levitan, Secretary-General, U.S.-side Panel

Remarks by Shyam Sunder, Director, Engineering Laboratory, National Institute of Standards and Technology, Department of Commerce

Remarks by Taketo Uomoto, Chairman, Japan-side Panel on Wind and Seismic Effects, Chief Executive, Public Works Research Institute

Introduction of U.S.-side Members by U.S.-side Panel Chairman

Introduction of Japan-side Members by Japan-side Panel Chairman

Adopt Agenda

Adjourn

9:05 Group Photograph

9:30 **Technical Session 1 - Lessons Learned from Recent Disasters: Design Codes and Standards Development**
Chairman - Dr. Taketo Uomoto

9:30 *Structural Design Requirement on the Tsunami Evacuation Buildings*, Hiroshi Fukuyama, Hiroto Kato, Tadashi Ishihara, Seitaro Tajiri, Masanori Tani, BRI; Yasuo Okuda*, Toshikazu Kabeyasawa, NILIM; and Yoshiaki Nakano, University of Tokyo

9:50 *Development of U.S. Structural Design Standards to Accommodate Natural Hazards*, James A. Rossberg* and James R. Harris, ASCE

10:10 *Outline of Japanese Design Specifications for Highway Bridges in 2012*, Tetsurou Kuwabara*, PWRI; Takashi Tamakoshi, NILIM; Jun Murakoshi, Yoshitomi Kimura,

	Toshiaki Nanazawa and Jun-ichi Hoshikuma, PWRI
10:30	*Establishing Design Criteria for All Extreme Loads (Multi-Hazard) for Transportation Infrastructure*, W. Phillip Yen*, FHWA; and George C. Lee, SUNY Buffalo
10:50	*Brief Review of Building Damage by the 2011 Tohoku Japan Earthquake and Following Activities for Disaster Mitigation*, Masanori Iiba*, Isao Nishiyama, Hiroshi Fukuyama, Izuru Okawa, BRI; and Yasuo Okuda, NILIM
11:10	*Assessment of Design Long-period Earthquake Motions for High-rise and Base-isolated Buildings*, Izuru Okawa*, BRI
11:30	Discussion
11:50	Lunch
12:40	**Technical Session 2 - Lessons Learned from Recent Disasters: Resilience Programs and Research**
	Chairman - Dr. Taketo Uomoto
12:40	*Disaster Resilience of Buildings, Infrastructure, and Communities*, Stephen A. Cauffman*, NIST
13:00	*3-Year Research Program on Risk and Crisis Management Strategy for Excessive and Multiple Actions of Natural Disasters*, Shigeki Unjoh* and Atsushi Hattori, NILIM
13:20	*Tsunami Resilient Ports on the Basis of Lessons from the 2011 Great East Japan Earthquake and Tsunami*, Takashi Tomita*, Taro Arikawa and Shigeo Takahashi, PARI
13:40	*Reinforced Concrete Wall Research Based on the Experience and Observations from February 2010 Maule, Chile, Earthquake*, Steven McCabe* and Travis Welt, NIST
14:00	*Drift Issues of Tall Buildings during the March 11, 2011, M9.0 Earthquake, Japan - Implications*, Mehmet Celebi*, USGS; and Izuru Okawa, BRI
14:20	Discussion
14:40	Break

15:00	**Task Committee Meetings**	
15:00	A: Strong Motions and Effects	[Lecture Room B]
	C: Dams	[Lecture Room C]
	D: Wind Engineering	[Portrait Room]
	G: Transportation Systems	[Lecture Room D]
16:00	B: Buildings	[Lecture Room B]
	C: Dams (Continued)	[Lecture Room C]
	G: Transportation Systems (Continued)	[Lecture Room D]
	H: Storm Surge and Tsunami	[Portrait Room]
17:00	Conclusion of Day 1	

February 21 (Thursday)

8:30	**Task Committee Reports** Chairman - Dr. John Hayes
9:30	**Strategic Planning Session** Chairmen - Drs. John Hayes and Taketo Uomoto
10:15	Break
10:30	**Technical Session 3 - Lessons Learned from Recent Disasters: Recent U.S. and Japan Field Activities** Chairman - Dr. John Hayes
10:30	*Technical Investigation of the May 22, 2011, Tornado in Joplin, Missouri*, Marc Levitan*, Frank Lombardo, Long Phan, Erica Kuligowski, NIST; and David Jorgensen, NOAA
10:45	*Mitigation Assessment Team Report, Spring 2011 Tornadoes*, John Ingargiola*, FEMA
11:00	*Report on Field Surveys and Subsequent Investigations of Building Damage Following the May 6, 2012 Tornado in Tsukuba City, Ibaraki Prefecture, Japan*, Yasuo Okuda*, Atsuo Fukai, Takahiro Tsuchimoto, Toshikazu Kabeyasawa, NILIM; Hitomitsu Kikitsu, Takafumi Nakagawa, Norimitsu Ishii and Yasuhiro Araki, BRI
11:20	*FIMA Briefing of the MAT Investigation of Hurricane Sandy*, John Ingargiola*, FEMA
11:40	*High Resolution Imagery Collection Utilizing Unmanned Aerial Vehicles (UAVs) for Post-Disaster Studies*, Stuart Adams, NIST/LSU; Marc Levitan*, NIST; and Carol Friedland, LSU
11:55	*Disaster and Failure Studies, Program and Data Repository*, Eric Letvin*, NIST
12:10	Discussion
12:25	Lunch
13:15	**Adoption of Final Resolutions** Chairman - Dr. John Hayes
14:10	**Closing Ceremonies** Call to order by Marc Levitan, Secretary-General, U.S.-side Panel Remarks by Taketo Uomoto, Chairman, Japan-side Panel on Wind and Seismic Effects Remarks by John Hayes, Chairman, U.S.-side Panel on Wind and Seismic Effects
14:30	Conclusion of 44th Joint Panel Technical Sessions

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RESOLUTIONS

RESOLUTIONS OF THE FORTY-FOURTH JOINT MEETING
U.S.-JAPAN PANEL ON WIND AND SEISMIC EFFECTS (UJNR)

National Institute of Standards and Technology, Gaithersburg, Maryland, USA
20-21 February 2013

The following resolutions are hereby adopted:

1. The Forty-Fourth Joint Panel Meeting provided the forum to exchange valuable technical information that is beneficial to both countries. In particular, both sides delivered presentations on the design codes and standards development and resilience programs in each country. In view of the importance of cooperative programs on the subject of wind and seismic effects, the continuation of Joint Panel Meetings is considered important. Both sides agreed to follow the recommendations of the Panel's Third Five-Year Strategic Plan and emphasize identifying opportunities, primarily through its Task Committees, for sharing and developing technologies that lead to new design and construction practices, and for providing users with improved design and construction procedures.

2. The following activities have been conducted since the Forty-Third Joint Meeting:

   a. Technology Exchanges. Technical experts and technical documents have been exchanged. These exchanges have contributed to the development of new research and enhanced ongoing research programs in both countries. In particular, the U.S.-side Panel members exchanged information with the Japan-side Panel members and supported them on the occasion of their investigation on the organizational arrangements for crisis management and crisis management technologies for natural disasters in the U.S.

   b. Task Committee Workshops. The Panel held four workshops:
      1) Task Committee G, 27th U.S.-Japan Bridge Engineering Workshop, 7-9 November 2011, Tsukuba, Japan
      2) Task Committee D, Workshop on Structural Dynamics and Monitoring of Bridges and Flexible Structures against Wind Hazards, 14 November 2011, Boston, MA, U.S.
      3) Task Committee D, Workshop on Structural Dynamics and Monitoring of Bridges and Flexible Structures against Wind Hazards, 11-13 March 2012, Lubbock, TX, U.S.
      4) Task Committee G, 28th U.S.-Japan Bridge Engineering Workshop, 8-10 October 2012, Portland, OR, U.S.

   c. Major Products. The Panel members produced or made significant contributions to advancing the Panel's mission:
      1) Task Committee C continued to conduct collaborative research on non-linear response analysis and discrete element method analysis of concrete dams.
      2) Japan side members of Task Committee H are participating with U.S. side Task Committee H members on the committee developing tsunami design

provisions for the ASCE 7 standard applicable to buildings and other structures.

3) A tornado simulator based on the design of Iowa State University Tornado Simulator was constructed at the Building Research Institute, Tsukuba, Japan, completed in 2011, under the supervision of Task Committee D member Dr. H. Kikitsu in collaboration with Task Committee D co-Chair Dr. P. Sarkar of ISU. The facility will be used to conduct research on tornado-induced wind loads at BRI.

d. The Panel members contributed to various efforts following the 2011 Great East Japan Earthquake.

1) Task Committee G initiated joint research on tsunami effects on bridge performance.
2) The Panel members of both sides discussed and exchanged information on tsunami effects on buildings in order to introduce tsunami loads into the building codes of both countries.
3) The Panel members of both sides conducted a joint reconnaissance of damage due to the 2011 Great East Japan Earthquake from August 31 through September 2, 2011.
4) Task Committee A members conducted joint research and published papers on data from buildings recorded during the 2011 Great East Japan Earthquake.

3. The Panel accepted the Task Committee reports presented during the Forty-Fourth Joint Panel Meeting. Each report included objectives, scope of work, accomplishments and future plans. Those reports are provided in a separate document.

4. The Panel conducted its Strategic Planning Session. Based on its Third Five-Year Strategic Plan, the Panel evaluated its accomplishments and Task Committees activities and made future recommendations. The following are the highlights.

a. The Panel functions in an umbrella role over its Member Agencies and Task Committees to enable smooth operation of the Panel. The Task Committees serve as the driving force of the substantial Panel activities.

b. The Panel explores further opportunities to work with academic researchers and professional and standards organizations. For example, Task Committee C extended invitations to the U.S. Society on Dams and the Japan Commission on Large Dams, to actively participate as members of the Task Committee. Task Committee C has incorporated several new members from these professional organizations. Another example is that the American Society of Civil Engineers (ASCE) can potentially contribute to joint standards development activities in the two nations.

c. The Panel holds its Joint Meetings with the core members of the both sides such as the Chairmen and Secretaries-General.

d. Joint Meetings may be held in conjunction with Task Committee meetings or workshops or other conferences.

e. The Panel continues to work toward streamlining its structure, encouraging and expanding the collaboration of researchers in both countries.

f. The results of the Panel's work should be widely disseminated to improve the quality of life globally. The Panel encourages greater use of e-mail, the Panel's eNewsletter, and the Panel's Web Site to share and disseminate data and information to Panel members and other researchers.

5. The Panel endorses the following proposed Task Committee Workshops during the coming year:

   a. Task Committee C, 5th U.S.-Japan Workshop on Advanced Research on Dams, August 2013, U.S.
   b. Task Committee G, 29th U.S.-Japan Bridge Engineering Workshop, October 2013, Tsukuba, Japan

   In the event that Task Committee co-chairs recommend conducting a joint meeting or workshop under the auspices of the UJNR Panel on Wind and Seismic Effects prior to the next annual meeting, that is not included in the above list, the Task Committee co-chairs will make a request to conduct the meeting through their respective Secretary-General for approval by the Joint Panel Chairmen.

6. The U.S. and Japan sides will plan, conduct, and share as appropriate, joint investigations following earthquake and wind disasters in the U.S., Japan and other countries.

7. The Forty-Fifth Joint Panel Meeting of the UJNR Panel on Wind and Seismic Effects will be organized as a core member meeting by the U.S.-side Panel, to be held in the U.S. in September or October 2013. The U.S.-side Secretariat will propose dates, meeting format, program, location, and itinerary with the concurrence of the Japan-side Panel.

STRATEGIC PLAN

ADDENDUM 2
Panel Expectations During 2011-2015

STRATEGIC PLAN
U.S.-JAPAN JOINT PANEL ON WIND AND SEISMIC EFFECTS

1. Introduction

This document is Addendum 2 of the Strategic Plan for the Panel on Wind and Seismic Effects, U.S.-Japan Cooperative Program in Natural Resources (UJNR), 2001-2005 (attachment). The Panel's 2001 Strategic Plan serves as the base of the Panel's operations and structure. This Addendum 2 provides a roadmap of outlined technical approaches for the Panel's operations during the next five-year period, 2011-2015. Addendum 2 provides minor modifications to the objectives presented in the Panel's original Charter, developed in 1987 at the 19th Joint Panel Meeting:

a. Encourage, develop, and implement the exchange of wind and earthquake engineering technology between appropriate U.S. and Japanese organizations to share scientific and technological knowledge.

b. Develop strong technical links of scientific and engineering researchers between the two countries and encourage exchanges of guest researchers.

c. Conduct joint research in areas of wind and earthquake engineering technology, including exchange of available research equipment and facilities in both countries, and publish findings from those efforts.

d. Conduct cooperative programs to improve engineering design and construction practices and other wind and earthquake risk mitigation practices, and publish results from those programs.

The Panel is worthy of continuation based on the fact that it is equipped with the following unique features:

1. UJNR is a government-to-government cooperation program in principle, and responsible information exchange between countries is possible, in terms of technical policies, design codes, technical standards and so forth. This also enables efficient reconnaissance surveys after a major disaster in either country, which was demonstrated in the case of the 2011 Great East Japan Earthquake.

2. The Panel is a comprehensive cooperation framework that covers wide areas such as transportation facilities, dams, buildings and ports. It is one of the Panel's advantages to be able to collect and integrate various opinions and views with relation to wind and earthquake engineering.

3. The built environments and construction processes in the two countries have many

similarities that make information sharing very productive, and the two countries have uniquely complementary research capabilities.

2. Panel Approaches During 2011-2015

The Panel's operational procedure is defined in the attached Strategic Plan (2001-2005). The Panel performs a self-evaluation during a Strategic Planning session held during its Joint Panel Meeting or other appropriate venues. Based on the Panel's evaluation, incremental modifications are carried out to enhance the Panel's operations and to bring 'value-added' to its users.

2.1. Panel Mission and Vision for 2011-2015

1. Continue performing post-disaster investigations and reconnaissance and sharing findings with Panel members and others. Panel members in the two countries recently performed these investigations and reconnaissance:

    a) Earthquakes: 2004 Niigata, 2004 Indonesia, 2005 Pakistan, 2007 Off Niigata, 2009 L'Aquila Italy, 2010 Chile, 2010-2011 Christchurch New Zealand, 2011 Great East Japan and others

    b) Typhoons and Hurricanes: 2005 hurricanes Katrina and Rita, 2004 Typhoon, 2008 Parkersburg and 2011 Joplin EF5 tornadoes.

2. Share U.S. and Japan National Disaster Mitigation Plan and Science and Technology Policy among the Panel members such as:

    a) USA:

        (1) NSTC/SDR: *Grand Challenge for Disaster Prevention- a 10-year strategy for disaster reduction through science and technology; Windstorm Impact Reduction Implementation Plan*.

        (2) NEHRP Strategic Plan (2009-2013).

    b) Japan:

        (1) Central Disaster Prevention Council (2005): *Reduction by half of human damage and economic damage in coming 10 years*.

        (2) Central Disaster Prevention Council (2008): *A comprehensive plan for achieving "no victim" from natural disasters*.

        (3) Reconstruction Headquarters in response to the Great East Japan Earthquake (2011): *A basic policy on reconstruction from the Great East Japan Earthquake*.

        (4) Cabinet Office (2011): *The 4th Science and Technology Basic Plan (2011-2016).*

3. Identify methods that support each country's efforts in disaster mitigation through cooperation between the U.S. and Japan and explore opportunities for joint research projects. This would include support for jointly sponsored post-disaster Rapid Response Research, for example, using the Rapid Research Response funding mechanism (RAPID) in the U.S. and J-RAPID mechanism in Japan and for sharing research finding from these investigations to further disaster mitigation.

4. During 2011-2015 the Panel will focus on topics such as:

   a) Study instrumented buildings that are similar to U.S. construction and that experienced significant motions, in view of the impacts of the 2011 Great East Japan Earthquake.

   b) Study the impact of long duration ground motions on the design of the built environment and in particular buildings, dams, bridges and ports.

   c) Pursue collaborative research to study the effects of the Tohoku tsunami on Japanese buildings, bridges and other structures that are similar to US construction, in order to enhance and modernize tsunami design codes.

   d) Facilitate exchange of technical information on typhoons, hurricanes and tornadoes.

   e) Continue to understand causes and effects of wind and seismic hazards and pursue the accumulation and interpretation of data.

   f) Evaluate and estimate risks of natural hazards.

   g) Improve/develop disaster mitigation technology and methodology, and dissemination of disaster response technology into practical applications.

   h) Promote increased research efforts that consider societal implications of natural disasters.

   i) Integrate technology development and the viewpoint of social/civil engineering by increasing the importance of the cooperative works between related UJNR Panels, the Panel's Task Committees, and the private sector and academia.

   j) Create methods to better integrate comprehensive technology information as a base for transmitting information throughout the Panel member's organizations. And,

   k) Contribute to dissemination of cooperative products that will facilitate global standardization of related civil engineering technologies.

2.2. Evaluate Task Committees. The Panel operates under six Task Committees, including one reformed on the occasion of the Forty-Third Joint Panel Meeting in 2011; an optimum number for Panel management and productivity. The Task Committees serve as the heart of the Panel's operations:

Task Committee A.	Strong Motions and Effects
Task Committee B.	Buildings
Task Committee C.	Dams
Task Committee D.	Wind Engineering
Task Committee G.	Transportation Systems
Task Committee H.	Storm Surge and Tsunami

Findings from Task Committee evaluations will help the committees: 1) measure achievements, productivity, and impact on contributions to improving design and construction practices, 2) identify opportunities for making contributions and addressing emerging technical challenges, and 3) assess when they completed their mission and are ready for retirement. Task Committee Evaluation Criteria include: 1) one or more workshops conducted at least every three to four years, 2) implementing recommendations from workshops, 3) publications and other outreach, and 4) collaborations beyond the specified Task Committee.

The Panel will encourage its respective Task Committees to identify thematic focuses requiring technology sharing and joint collaborations. These themes will be discussed at Panel Meetings. The Panel will consider the merits of creating new Task Committees that meet special needs and eliminating Task Committees that have completed their missions or can be strengthened through consolidation with other Task Committees.

2.3. <u>Partnering Opportunities.</u> Identify partnering opportunities through joint activities of appropriate Panel's Task Committees, collaborating with other UJNR Panels, and working together with the private sector and academia. Joint activities optimize Task Committee resources (human and financial). Partnering activities will be discussed at Panel Meetings, including increasing participation from the private sector and academia.

2.4. <u>Joint Research.</u> Perform joint research initiated by the Panel and its Task Committees. Task Committees are encouraged to identify key joint-research opportunities to improve the state-of-knowledge or to consider engaging in a significant long-term research funded from one or more sponsoring organizations. For the latter, below are the Panel Cooperative Research Projects performed during the past 32-years that improved design and construction practices for both countries:

1. Reinforced Concrete Structures (1979-1987): accomplishments include testing six-story full scale buildings which led to improve seismic design methods of reinforced concrete buildings.

2. Seismic Performance of Lifeline Facilities (1982-1989): accomplishments include development of improved seismic design methods of bridge columns.

3. *In-situ* Testing Methods for Soil Liquefaction (1983-1986): accomplishments include development of rationale for Standard Penetration Test (SPT) data based on energy ratio.

4. Masonry Structures (1984-1988): accomplishments include development of strength-based design guidelines for reinforced masonry buildings.

5. Steel-Frame Structures (1985-1987): accomplishments include testing of a full-scale five-story building to confirm prediction of performance based on components and subassemblages.

6. Bridge Hybrid Control Systems (1990-1994): accomplishments include development of hybrid control algorithms that require less energy for controlling bridge response.

7. Precast Seismic Structural Systems (1991-1992): accomplishments include development of strength-based design guidelines.

8. Seismic Performance of Composite and Hybrid Structures (1993-1998): accomplishments include development of design guidelines for composite and hybrid system, and development of new materials.

9. Countermeasures for Soil Liquefaction (1994-2004): accomplishments include contributions on the revision of design guidelines for building foundations and formulation of soil experiment plans using E-Defense.

10. Development of Smart Structural Systems (1998-2003): accomplishments include development of structural performance detection technology and structural members using intelligent materials.

11. Develop Comparative Analysis of Seismic Performance Testing Guidelines for Bridge Piers (1999-2006): accomplishments include a joint publication on the comparative analysis of US and Japan bridge piers.

12. Flutter Derivatives on Bridge Girders (2002-2006): accomplishments include comparisons of flutter derivatives on bridge girders by US and Japan wind tunnel tests.

13. Wind Effects on Typical Low-Rise Industrial Buildings (2003-2008): accomplishments include comparisons of wind pressures on low-rise buildings by US and Japan wind tunnel tests.

14. Non-linear Response Analysis and Discrete Element Method Analysis of Concrete Dams (2006-present).

15. Study of Tornadic Flow and Effects on Buildings Structures (2010-present).

During the period of 2011-2015, the Task Committees are encouraged to commence new joint research activities. Possible themes include, but are not limited to:

1. Lessons learned from instrumented structures whose responses were recorded during the

2011 Great East Japan Earthquake and aftershocks (Task Committee A).

2. Coordinated research regarding structural wall performance (Task Committee B).

3. Strategy to determine design criteria, design loads, and load factors that consider ductility and redundancy for multiple hazards (Task Committee G).

4. Study on policy making to set different performance levels of routes and allocate resources for seismic upgrading/retrofit, bridge inspection, and rehabilitation based on the assigned characteristics (Task Committee G).

5. Study on tsunami damage estimation in modern coastal cities (Task Committee H).

The respective Task Committees are asked to identify candidate joint research for discussion at Panel meetings. For each joint research project that is proposed, the respective Task Committees will designate appropriate lead investigators for both sides.

2.5. Panel Communications. The Panel will explore the feasibility of more broadly disseminating Panel's activities, accomplishments, and impacts including findings from post-disaster investigations using the Panel's eNewsletter, a more active Web Site, Task Committee publications, and identify Panel accomplishments and impacts. The Panel will explore increased information sharing among its member organizations and include links to related organizations in both countries. Task Committees can serve as a knowledge base of information on their respective themes and share their information to users following methods described above.

2.6. Joint Panel Meeting Format. The Panel has reorganized its Joint Meeting format since the Forty-Third Joint Panel Meeting in 2011. The Panel holds two types of Joint Panel Meetings alternately: Joint Panel Meetings for the Chairmen and Secretaries-General of the both sides and other interested parties, and full Joint Panel Meetings.

3. Conclusion

This Addendum represents the Panel's focus to address panel's activities of the next five-years. The strategic plan is evaluated during its Meetings.

ADDENDUM 1
Panel Expectations During 2006-2010

STRATEGIC PLAN
U.S.-JAPAN JOINT PANEL ON WIND AND SEISMIC EFFECTS

1. Introduction

This document is Addendum 1 of the Strategic Plan for the US-Japan Panel on Wind and Seismic Effects 2001-2005 (attachment). The Panel's 2001 Strategic Plan serves as the base of the Panel's operations and structure. This Addendum 1 provides a roadmap of outlined technical approaches for the Panel's operations during the next five-year period 2006-2010. As background, the Panel's Charter, developed in 1987 at the 19th Joint Panel Meeting, is to:

a. Encourage, develop, and implement the exchange of wind and seismic technology between appropriate US and Japanese organizations to share scientific and technological knowledge.
b. Develop strong technical links of scientific and engineering researchers between the two countries and encourage exchanges of guest researchers.
c. Conduct joint research in areas of winds and seismic technology including exchange of available research equipment and facilities in both countries. Publish findings from joint research efforts.
d. Conduct cooperative programs to improve engineering design and construction practices and other wind and earthquake hazard mitigation practices. Publish results from cooperative programs.

2. Panel Approaches During 2006-2010

The Panel's operational procedure is defined in the attached Strategic Plan (2001-2005). Annually the Panel performs a self-evaluation during a Strategic Planning session held during its May Joint Panel Meeting. Based on the Panel's evaluation, incremental modifications are carried out to enhance the Panel's operations and to bring 'value-added' to its users.

2.1. <u>Panel Mission and Vision for 2006-2010</u>

1. Continue performing post disaster investigations and reconnaissance and sharing findings with Panel members and others as was carried out for:
   a) Earthquakes: (2004 Niigata, 2004 Indonesia, 2005 Pakistan, and others)
   b) Typhoons and Hurricanes: (2005 Katrina and Rita, 2004 Typhoon)
2. Share US and Japan National Disaster Mitigation Plan among the Panel members such as:
   a) USA. NSTC/SDR: *Grand Challenge for Disaster Prevention- a 10-year strategy for disaster reduction through science and technology*; *Windstorm Impact Reduction Implementation Plan*; the NEHRP Annual Plan.
   b) Japan. *Central Disaster Prevention Council: Reduction by half of human damage and economic damage in coming 10 years*, Technology Development Plan at "Council for Science and Technology"
3. Identify methods that support each countries efforts in disaster mitigation through cooperation between the US and Japan and explore opportunities for joint research projects.

4. During 2006-2010 the Panel will focus on topics such as:
   a) Continue to understand causes and effects of wind and seismic hazards and pursue the accumulation and interpretation of data
   b) Evaluate and estimate risk of natural hazards
   c) Improve/develop disaster mitigation technology and methodology, and dissemination of disaster response technology into practical applications
   d) Promote attention to increase research that considers societal implications of natural disasters
   e) Integrate technology development and the viewpoint of social/civil engineering by increasing the importance of the cooperative works between related UJNR Panels, the Panel's Task Committees, and the private sector and academia
   f) Create methods to better integrate comprehensive technology information as a base for transmitting information throughout the Panel member's organizations
   g) Contribute to dissemination of cooperative products that will facilitate global standardization of related civil engineering technologies

2.2. Evaluate Task Committees. The Panel operates under seven Task Committees; an optimum number for Panel management and productivity. The Task Committees serve as the heart of the Panel's operations:

Task Committee A.	Geotechnical Engineering and Ground Motion
Task Committee B.	Next Generation Building and Infrastructure Systems
Task Committee C.	Dams
Task Committee D.	Wind Engineering
Task Committee G.	Transportation Systems
Task Committee H.	Storm Surge and Tsunami
Task Committee I.	Fire Performance of Structures

Findings from Task Committees' evaluations will help the Task Committees 1) measure achievements, productivity, and impact on contributions to improving design and construction practices, 2) identify opportunities for making contributions and addressing emerging technical challenges, and 3) assess when they completed their mission and are ready for retirement. Task Committee Evaluation Criteria includes: 1) one or more workshop conducted at least every three-years, 2) implementing recommendations from workshops, 3) publications and other outreach, and 4) collaborations beyond their Task Committee.

The Panel will encourage its respective Task Committees to identify thematic focuses requiring technology sharing and joint collaborations. These Themes will be discussed at annual Panel Meetings. The Panel will consider the merits of creating new Task Committees that meet special needs and eliminating Task Committees that have completed their mission or can be strengthened through consolidation with other Task Committee(s).

2.3. Partnering Opportunities. Identify partnering opportunities through clustering appropriate Panel's Task Committees, collaborating with other UJNR Panels, and working together with the private sector and academia. Clustering provides Task Committee optimization of resources (human and financial). Partnering and clustering will be discussed at annual Panel Meetings including increasing participation from the private sector and academia.

2.4. Joint Research. Perform joint research initiated by the Panel and its Task Committees. The Task Committees are encouraged to identify key joint-research opportunities to improve the state-of-knowledge or to consider engaging in a significant long-term research funded from one or more sponsoring organizations. For the latter, below are the Panel Cooperative Research Projects performed during the past 27-years that improved design and construction practices for both countries.

1. Reinforced Concrete Structures (1979-1987); accomplishments include testing six-story full scale buildings which led to improve seismic design methods of reinforced concrete buildings.
2. Seismic Performance of Lifeline Facilities (1982-1989); accomplishments included development of improved seismic design methods of bridge columns.
3. *In-situ* Testing Methods for Soil Liquefaction (1983-1986); accomplishments include development of rationale for Standard Penetration Test (SPT) data based on energy ratio.
4. Masonry Structures (1984-1988); accomplishments include development of strength-based design guidelines for reinforced masonry buildings.
5. Steel-Frame Structures (1985-1987); accomplishments include testing of a full-scale five-story building to confirm prediction of performance based on components and subassemblages.
6. Bridge Hybrid Control Systems (1990-1994); accomplishments include development of hybrid control algorithms that require less energy for controlling bridge response.
7. Precast Seismic Structural Systems (1991-1992); accomplishments include development of strength-based design guidelines.
8. Seismic Performance of Composite and Hybrid Structures (1993-1998); accomplishments include development of design guidelines for composite and hybrid system, and development of new materials.
9. Countermeasures for Soil Liquefaction (1994-2004); accomplishments include contributions on the revision of design guidelines for building foundations and formulation of soil experiment plans using E-Defense.
10. Development of Smart Structural Systems (1998-2003); accomplishments include development of structural performance detection technology and structural members using intelligent materials.
11. Develop Comparative Analysis of Seismic Performance Testing Guidelines for Bridge Piers (1999-2006); accomplishments included a joint publication on the comparative analysis of US and Japan bridge piers.

The respective Task Committees will identify candidate joint research for discussion at annual Panel meetings.

2.5. Panel Communications. More broadly disseminate Panel's activities, accomplishments, and impacts including findings from post-disaster investigations using the Panel's eNewsletter, a more active Web Site, Task Committee publications, and identify Panel accomplishments and impacts. The Panel will increase information sharing among its member organizations and include links to related organizations in both countries. Task Committees will serve as a knowledge base of information on their respective themes and share their information to users following methods described above.

3. Conclusion

This Addendum represents the Panel’s focus to address panel’s activities of the next five-years. The strategic plan is annually evaluated during its annual May Meetings.

U.S.-Japan Joint Panel on Wind and Seismic Effects
Strategic Plan

1. Introduction

1.1 Context

The U.S. and Japan must maintain an awareness of international developments in earthquake and wind mitigation technology. The international exchange of information is achieved through a combination of formal and informal mechanisms, including: attendance at conferences and workshops; cooperative research projects and programs; and exchange of scientists and engineers. There is a long-established tradition of joint research activities between Japan and the United States. The U.S.-Japan Cooperative Program in Natural Resources (UJNR) Panel on Wind and Seismic Effects (WSE Panel) provides a formal government-to-government mechanism for cooperation between the two countries in the area of earthquake and wind mitigation technology.

At the 32nd Joint Panel Meeting, a resolution was passed to establish a joint Ad-Hoc Committee for the purpose of developing a strategic plan for the WSE Panel. The catalyst for this effort was the need to address immediate issues related to cost and participation. While the Panel recognized the importance of addressing these immediate issues, it also realized that an opportunity existed to strengthen the WSE Panel's focus on its core mission and foster greater collaboration between researchers in the U.S. and Japan while streamlining the overall operation of the Panel. It was with this goal in mind that the ad-hoc committee developed the strategic plan contained in this document.

1.2 Approach

Before meeting to develop the strategic plan, each side held domestic panel meetings and conducted one-on-one meetings with participating agencies to identify issues that needed to be addressed by the strategic plan and to understand which features of the Panel and its operation should be retained and which needed to be changed or adapted to meet current and future needs. Each side developed a concept paper to capture these ideas. The concept papers, however, tended to focus on addressing the immediate issues rather than positioning the Panel to address the needs and challenges of the future. Through the exchange of the concept papers and subsequent discussion, the two sides moved close to agreement on near-term changes to the Panel's operation. Thus, the strategic plan emphasizes longer-term goals for the operation and growth of the Panel and a time-phased approach to implementation of steps to achieve these goals.

The strategic plan is intended to establish a course for the WSE Panel over the next 5-10 years. It recognizes that there are many ways in which the Panel may work to achieve the goals identified, and so while some steps in the implementation process are clear, others are left open to be determined through experimentation. However, the Panel believes working toward the goals identified will strengthen its role in engineering and scientific communities of the U.S. and Japan and will allow our countries to make more efficient use of resources to conduct research and disseminate results to the benefit of both countries.

2. Role of the Panel

2.1 Guide Research Agendas

As a government-to-government mechanism for collaboration, the WSE Panel is in a unique position to guide the development and execution of each country's research agenda. Currently, each country defines its own research priorities, projects are formulated in a fragmented manner, and results are reported through vehicles such as the Annual Joint Panel Meeting. By strengthening its ties to industry and academia, the Panel will be able to identify specific research needs and align those with government priorities. The Panel shall work toward a coordinated research agenda that permits the efficient use of human resources, funding, and research facilities to achieve mutual research objectives.

The new Task Committees formed through implementation of this strategic plan shall work to identify areas where joint research projects can be established and conducted as a part of a coordinated research agenda. Joint research projects may include participation by university or industry researchers in addition to member agency researchers.

2.2 Leverage Resources

The U.S. and Japan each possess significant expertise in the fields of earthquake and wind engineering and have a substantial investment in equipment and facilities to perform testing and measurements in support of research in these fields. Historically, the WSE Panel has facilitated the exchange of researchers between the U.S. and Japan but has not made a concerted effort to leverage the resources of the two countries. There is an opportunity for the Panel to coordinate research activities to efficiently utilize testing and measurement facilities in both countries to address mutual research needs and avoid duplication. This is an area in which the Panel can significantly strengthen its efforts and provide a tangible benefit to each country by working to establish strong partnerships for coordinated research.

2.3 Foster Cooperation

From its founding, the WSE Panel has promoted cooperation between the U.S. and Japan through annual Joint Panel Meetings, Task Committee activities, and exchange of researchers. One of the hallmarks of success for the WSE Panel through the years has been the high level of cooperation. The model these cooperative efforts have been built around, however, is one of information exchange. While the exchange of information and research results is an important facet of the WSE Panel's work, there is the opportunity to greatly expand the scope and importance of cooperative efforts to leverage resources (people, funding, facilities) through joint research projects of bilateral importance. Additionally, the Panel should look beyond the government agencies that participate to be more inclusive of universities and the private sector. At a minimum, this will include broadening participation in the Joint Panel Meetings to include industry and university participants. As industry and universities become more engaged, exchanges of researchers among government, university, and industry participants may be possible. Joint programs that include participation by government, industry, and university partners shall also be considered. These activities would broaden the reach of the Panel and provide a means for more rapid diffusion of research results into practice.

2.4 Technical Exchange

The WSE Panel has, throughout its history, been an effective mechanism for the exchange of technical information between Japan and the United States. Further, the WSE Panel has provided a means for disseminating measurements and research results to other nations affected by earthquake, wind, tsunami, and storm surge hazards. Annual Joint Panel Meetings, Task Committee Meetings and Workshops, researcher exchange programs, and personal relationships among researchers have fostered this exchange. The Panel recognizes this as one of its strengths and should seek to broaden its reach to include participation by researchers in other nations. The Panel should explore means of increasing collaboration with other countries through inclusion of representatives from other nations in Joint Panel Meetings, encouraging joint projects through the Task Committees that include partners outside the U.S. and Japan, and through the exchange of researchers with other countries.

2.5 Engage Private Sector

The WSE Panel has engaged the private sector to a limited extent in its activities during its history, although the work of the Panel and the participating agencies can have a direct benefit to industry and ultimately the public in our respective countries. Further, some larger companies have research capabilities and programs that could enrich the Panel. More actively engaging the private sector will provide a means for obtaining input in setting priorities and for more rapidly diffusing the results of research activities into practice within Japan and the United States. The involvement of the private sector may include participation in the development of coordinated research agendas and dissemination of information perhaps through special sessions at the annual Joint Panel Meetings. The Panel should also consider involving the private sector in research projects coordinated at the Task Committee level that will have broad-based benefits to industry in both countries. Longer-term, the involvement of the private sector will facilitate dialogue between practicing engineers and builders in Japan with their counterparts in the U.S. The Joint Panel will examine ways to increase industry participation, initially by inviting key industry representatives to participate in Joint Panel Meetings and to speak about the work of their company or organization and explore possibilities for greater collaboration.

2.6 Web Page Development

The Joint Panel will explore ways to increase utilization of the Internet as a means of communication both among Panel members and with outside organizations. The Japan-side has offered to take the lead in developing an Internet presence for the Joint Panel and has begun work on an initial concept for the site. Once the site is established, the Task Committees will be relied on to provide, maintain, and update content related to their activities. The Joint Panel will also explore ways of using Internet resources as a means of facilitating communication among researchers as well as the exchange and dissemination of information and research results.

3. Implementation

3.1 Strategic Plan Development and Approval

This strategic plan was prepared through the efforts of the Joint Ad-Hoc Committee appointed by the Chairmen following the 32nd Joint Panel Meeting. The Joint Panel shall work toward approval of the Strategic Plan during the 33rd Joint Panel Meeting May 28-30, 2001. The approved document will reflect comments received from Panel members on the draft. Implementation of the strategic plan will begin with approval and require 12-24 months for full implementation.

3.2 Task Committee Charters and Recommended Committees

The US- and Japan-side Panels have agreed on the following seven themes around which Task Committees may be formed:

Theme A: Geotechnical Engineering and Ground Motion
Theme B: Buildings
Theme C: Dams
Theme D: Wind
Theme E: Lifelines
Theme F: Seismic Information and IT
Theme G: Transportation
Theme H: Storm Surge and Tsunami
Theme I: Public Health

Task Committee formed to address one of these themes will be approved by the Joint Panel on annual basis, provided that the Task Committee remains active. The criteria for active Task Committees are following:

a) Conducts joint workshops or technical meetings on a regular basis for the purpose of exchanging technical information, research results, or data for the mutual benefit of both countries.
b) Engages in frequent exchange of researchers for the purpose of technical interchange and collaboration on research.
c) Conducts one or more joint research projects having clearly defined technical objectives, finite duration, and shared responsibility for producing technical results.

Task Committees will report results through papers presented during the joint panel meeting and through task committee reports. The Joint Panel will review task committee results and future plans on an annual basis and will approve task committees for the next year based on this information.

For this year, new task committees may be established by requesting approval through the Secretary-Generals at any time before the 34th Joint Panel Meeting.

3.3 Transition to New Annual Panel Meeting Format

A number of alternative formats for the annual Joint Panel Meeting were considered. Based upon the input received from Panel members, the basic format of the Joint Panel Meeting be retained. Session topics will be principally driven by the Task Committees. Each Task Committee would be given one session during which it would be able to present research results. This Task Committee-driven format should strengthen the role of the Task Committees and is intended to stimulate greater cooperation among researchers in each country. This format will foster the exchange of information that many have expressed is a desirable feature of the Joint Panel Meeting.

The Joint Panel meeting will be shortened by one day (from 4 days to 3 days). The shorter meeting, coupled with a shorter Technical Site Tour will reduce the time commitment for participants to one week. This is intended to encourage greater participation in the Panel Meetings and Site Tours, particularly by members of the visiting Panel.

Finally, the Joint Panel will explore streamlining the Joint Panel Meeting to maximize the opportunity for technical exchange.

The shortened Panel Meeting/Technical Site Tour format is implemented for the first time at the 33rd Joint Panel Meeting. The Task Committee-driven technical meeting format will be implemented at a later date when the revised Task Committee organization is in place.

4. Conclusion

The plan outlined above represents a strategic plan for positioning the Joint Panel to meet the challenges of the future, while retaining those aspects that have contributed to its success through it 32 year history. This plan is intended to address the current realities of the Panel, as well as increase the value and contribution of the Panel to the U.S. and Japan. Full implementation of the strategic plan will take approximately two years.

PAPERS

3-Year Research Program on Risk and Crisis Management Strategy for Excessive and Multiple Actions of Natural Disasters

by

Shigeki Unjoh[1] and Atsushi Hattori[2]

ABSTRACT

NILIM has started a new 3-year research project on "Risk and Crisis Management Strategy for Excessive and Multiple Actions of Natural Disasters (EMAND)" from 2012. Based on the lessons learned from the destructive damage caused by the 2011 Great East Japan earthquake, the research objective of the project is to improve Japan's emergency management capability (preparedness and responses) against extreme natural disasters including earthquake, heavy rain, flood, volcano, slope failure and the complex. The key words of the research is "Think outside the Box," and "Beyond the Estimations," as well as "Not Only Disaster but the Complex Disasters," to include concepts and thoughts to respond the disaster phenomenon with low frequency but high impacts. This paper presents the outline of the project including objectives, research issues and target outcomes.

KEYWORDS: Emergency Management, Excessive and Multiple Actions of Natural Disasters (EMAND), Preparedness, Research Project, Response, Risk and Crisis Management

1. INTRODUCTION

During Great East Japan earthquake of March 11, 2011, the giant tsunami which exceeded the past estimation and the strong shaking attacked to the Pacific sea side of Tohoku to Kanto regions, and caused catastrophic damage in wide area over the length of 500km. After the 5 months of the earthquake, the typhoons of #12 and #15 also attacked the affected area and the flooding and sediment disaster followed on the earthquake. Thus, the damage and impacts became complex phenomenon.

Important lessons learned from the earthquake and the following flood disasters are as:

1) Improvement of preparedness to natural disasters which exceed past experiences and the estimation
2) Preparedness for effects of multiple and combined actions of natural disasters including earthquake, tsunami, heavy rain, flood, slope failure and so on.

These aspects were pointed out in the Committee for Technical Investigation on Countermeasures for Earthquakes and Tsunamis Based on the Lessons Learned from the "2011 off the Pacific coast of Tohoku Earthquake," the Central Disaster Management Council [1]. The committee issued the following recommendations in September 28, 2011.

1) Consider scientifically possible maximum scale of earthquake and tsunami
2) Recognize possibility of uncertainty in the estimation
3) Conduct every possible efforts for disaster mitigation to expected trench-type and inland-type earthquakes
4) For the damage assessment, consider combined movement of estimated fault zones and the occurrence time (day or night time), and the combination with typhoons

Reconstruction Headquarters in response to the Great East Japan Earthquake, Government of Japan, developed "Basic Guidelines for Reconstruction in response to the Great East Japan Earthquake," and issued it on July 29, 2011. The basic policies are the followings.

1) To make resilient and disaster-resistant society, put every possible effort based on "mitigation/evacuation" concept rather than "prevention"

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2) Strengthen national land to prepare large scale disasters

3) Make tough and ductile multiple defense concept

Based on the lessons learned from the destructive damage caused by the 2011 Tohoku earthquake, NILIM has started a new 3-year research project on "Risk and Crisis Management Strategy for Excessive and Multiple Actions of Natural Disasters (EMAND)" from 2012. The research objective of the project is to improve Japan's emergency management capability (preparedness and responses) against extreme natural disasters including earthquake, heavy rain, flood, volcano, slope failure and the complex. The key words of the research is "Think outside the Box," and "Beyond the Estimations," as well as "Not Only Disaster but the Complex Disasters," to include concepts and thoughts to respond the disaster phenomenon with low frequency but high impacts. This paper presents the outline of the project including objectives, research issues and target outcomes.

2. OBJECTIVES OF THE RESEARCH

The objective of the research is to clarify, define and measure the occurrence and the impact of the excessive and multiple actions of natural disasters which had not been considered in the past, and to propose the technology to enhance and improve the emergency management structures.

As shown in **Fig. 1**, there are several types of hazards including earthquake, tsunami, flood, volcano, storm surge and slope failure. The countermeasures have been constructed to treat each disaster independently in the past, the target of the research is to cover the area of the possibility to exceed the expected level (design level) and the combined impacts of multiple disasters. Foe example, complex action includes the flooding after the earthquake (as the case of 2011 Great East Japan Earthquake), and sediment and flooding disaster caused by the rainfall after the volcano eruption.

For these objectives, the research focuses on the following 3 points.

1) To know fully about EMAND
2) To see through EMAND, and
3) To manage coolly EMAND

Fig. 2 shows the illustration of countermeasures to EMAND.

3. RESEARCH ISSUES

3.1 Collection and Re-Analysis of Past Disasters

To know fully EMAND, the data of the past disasters and damage experiences is now being collected and re-analyzed. The different points from the past research are to learn again what happened and to think what will happen even though there has been no actual experience. Approach to consider both actual events and possible events with no experience is used for the analysis.

The events of disaster and the chain spreading influences to the society are to be studied using tree diagram concept. **Fig. 3** shows just a simple example of tree diagram. The earthquake occurs, and the disaster events including tsunami, liquefaction, ground settlement and cracks and sliding are caused. And such events furthermore cause next events and increases risks. The research focuses on clarification of disaster mechanism again because these approaches can enhance the capability not to miss all possible important disaster events and can be the base of flexible disaster management with well understanding of disaster phenomenon.

Past disasters to be analyzed include the 2011 Great East Japan Earthquake, 2012 Hurricane Sandy in US, and other recent large disasters in and out of Japan. Attention is also paid to the historical disasters to be analyzed. **Fig. 4** shows examples of historical disasters occurred in Japan. One is the Ansei earthquake in 1858. Large landslides occurred and developed the landslide dam. The dam failed by the aftershock, then flooding attacked the residence area. The other is Tenmei eruption of the Asama mountain in 1783. Pyroclastic flow developed sediment dam and the dam failed. In 1786, 3 years after the eruption, over 1000 people were killed by the volcanic mud flow. Social and economical situation when such historical disaster happened is completely different from the current one, but the attention is just paid to every experience.

3.2 Development Methods of Scenarios and Evaluation of Risk and Impact

To see through EMAND, scenario based evaluation methods for disaster occurrence and its chain progress is being studied as well as the risk evaluation methods to measure the size and the importance of the risk of events quantitatively. It is essential to understand how large the effect of the disaster and what must be protected and what is effective countermeasures to minimize the effects.

Based on the re-analysis of actual disasters as described in the above, the method to build scenario structure of events dependent on the social and economical conditions of the regions, which caused by the earthquake, heavy rain, volcano and others as well as the combinations, is being studied. As a typical model region is assumed to include mountain, flat, coastal, and urban areas. The social characteristics such as the population and the industry must be considered.

Based on these scenarios, the size of risk/influence is computed to evaluate the disaster quantitatively. The evaluation key index is assumed to be casualty and direct/indirect damage cost. The evaluation methods of risk and the influence should consider the mitigation effect by the applied countermeasures.

It is essential for the development of the scenario to setup the chain spreading of the damage and the events. **Fig. 5** shows a simple image of impact chain spreading of the damage of infrastructures to the society. The damage to roads, for example, causes the disfunction of the roads and results in the delay of recovery activities and the lack of emergency goods and equipment, then affects the society. For example, in the case of flooding, the inundation to underground spaces causes the fail to escape of people there. The power down caused by the water is also one of the important chain spreading mechanisms. During the 2011 Great East Japan earthquake, the lack and shortage of the gasoline, which caused by the damage to the oil refineries in the area, was also one of the biggest chain spreading.

Thus, the developed disaster scenario for a model region, the impact size by the index of casualty and/or damage costs is evaluated dependent on the level of the excessive and multiple actions of natural disasters. **Fig. 6** shows an example of evaluation index of disaster risk and impacts proposed by the USACE. The relation between the number of casualty and the occurrence probability of the casualty is shown. From **Fig. 6**, the occurrence probability to cause casualty of 100 is about one millionth. Aspects on the acceptable risk for the public and the cost of countermeasures to minimize casualty is necessary to be studied.

3.3 All-out Mobilization Concept of Countermeasures

To manage coolly EMAND, based on the scenario and the impact index as described in the above, the effective and possible countermeasures to minimize the risk and impacts, and to hold back the chain spreading with large effects are studied. The combination of the prevention, mitigation, and evacuation countermeasure concept, and the multi-level protection concept depending on the level of disasters are studied. The countermeasures include the provision and improvement of disaster prevention/mitigation infrastructures, and the improvement of emergency management system to respond to the disasters. The multi-level countermeasure concept means that the upper limit of hazard level may not be considered. **Fig. 7** shows the multi-level disaster response concept dependent on the level of disasters. When the level of hazard becomes larger, the disaster impact also increases, and the countermeasures and the response policy also should be changed to be appropriate to the level of disasters. The impact index curve for a big city area and a rural local area are shown in **Fig. 7** as examples. To consider the disasters without upper limit, it is an important point to recognize the inflection point that the impact index remarkably increases with the increase of hazard level dependent on the regional characteristics. Based on such analyses, it is possible to recognize what should be protected primarily dependent on the level of hazards. Even when the disaster exceeded the expected level, it is essential to always provide the next move.

Fig. 8 shows just an example of the menu of countermeasures. This shows a matrix of countermeasures before and after the events, and of those by the provision of protective infrastructures from the disasters and by the planning and information to protect human lives. One of the research targets is to provide the menu of effective measures. The tree diagram as shown in **Fig. 5** is to be used to find out what the efficient method to stop the chain spreading is. Risk curve as shown in **Fig. 6** is to be used for the development of disaster prevention infrastructures and for the quantitative evaluation of the selection of the countermeasure menu. The efficient methods are selected within the acceptable risk and the necessary costs to do those.

4. TARGET OUTCOMES

The research has been got down to from last April. The target of outcomes is the "Development of Countermeasure Guidelines on Disaster Mitigation Technology Application." The guidelines include the standard scenario development method and the impact index evaluation method to consider EMAND, as well as the countermeasure menus based on the multi-level disaster response concept. The guidelines are expected to be one of useful references for the application of the disaster response planning and the selection of the countermeasures. It is expected to be used for the assessment of the disaster capability of local regional and national levels. The important point is to extend the idea of outside the box and beyond the estimation not to limit a certain level. It is also important issue to know the limit of the protection infrastructure and provide the next move even if the provided protection is failed. The combination with the information and evacuation technology to save lives, and the quick function recovery just after the events and the systematic permanent rehabilitation technology are also essential. Such technology development and improvement will result in the development of the resilient society. The outcomes are expected to apply for the rehabilitation from the Great East Japan earthquake and to prepare next events including the Nankai Trough Giant Earthquake with M9.

5. CONCLUSIONS

The research project has been conducted jointly by Research Center for Disaster Management and River Department of NILM. To make the research outcome into practice, the research will be made in cooperation with MLIT Headquarters and Regional Bureaus. NILIM will try to cooperate or exchange technical information with academic community and foreign countries through the channels including the US-Japan Panel on Wind and Seismic Effects, UJNR, in order to make results be informative considering a wide area of disasters with low frequency and high impact.

It should be note here that the project is included in the MLIT's Master Plan for Technology Development which was issued by the Panel on Infrastructure Development on Dec. 10, 2012. The MLIT master plan is to show the policy direction of technology development and the efforts on R&D to be proceeded with a high priority.

6. REFERENCES

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ACKNOWLEDGEMENTS

The first author visited US government agencies in Dec. 2-8 in 2012 to learn the US practices to manage natural disasters including the advanced system/technology for such disaster management that has been remarkably improved based on the experiences during the 2005 Hurricane Katrina and others. The authors greatly thank Dr. W. Phillip Yen, Principal Bridge Engineer, Office of Bridge Technology, FHWA/DOT, who kindly arranged all the visits to US federal agencies and Louisiana state agencies during the visit. Great appreciation goes to Dr. Steven L. McCabe, NIST, Mr. Steven Cauffman, NIST, Mr. Dan Ferezan, FHWA/DOT, Ms. Sheila Duwadi, FHWA/DOT, Mr. Lois Triandafilou, FHWA/DOT, Mr. Hossein Ghara, LADOTD, Mr. Christopher Guilbeaux, GOHSEP, Mr. Michael Park, USACE, Prof. Brian Wolshon, LSU, Mr. Richard Swan, LADOTD, Mr. Bill Icenogle, LADOTD, Mr. Artur D’Andrea, LADOTD, and Ms. Jenny Fu, LADOTD, for providing us the useful information on the US practices during their busy time.

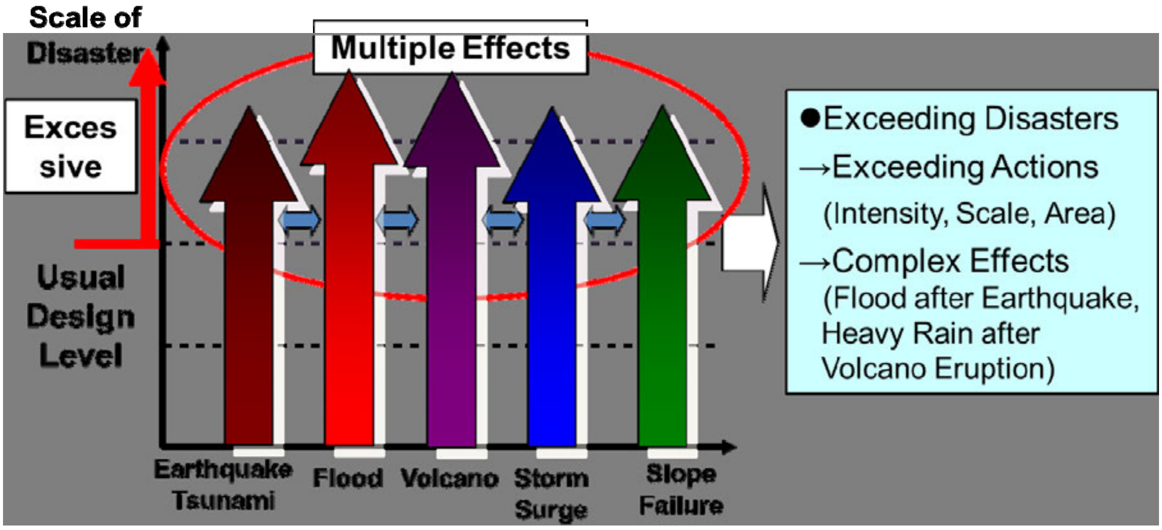

Fig. 1 Excessive and Multiple Actions of Natural Disasters

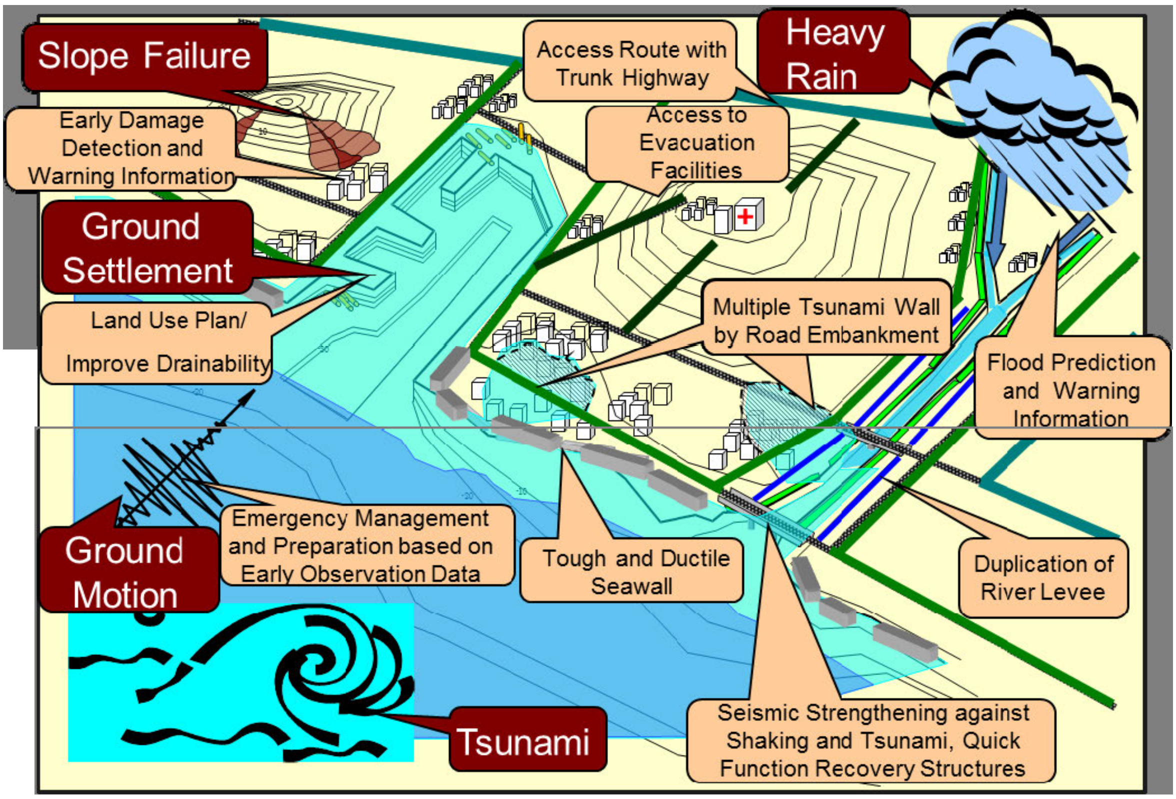

Fig. 2 Illustration of Countermeasures to Excessive and Multiple Actions of Natural Disasters

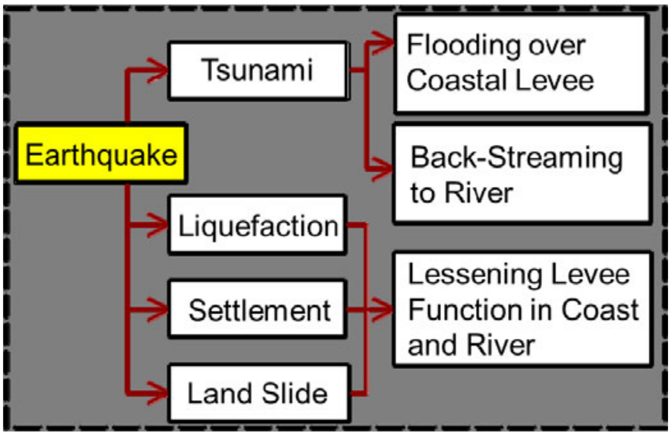

Fig. 3 Damage and its Chain Impacts

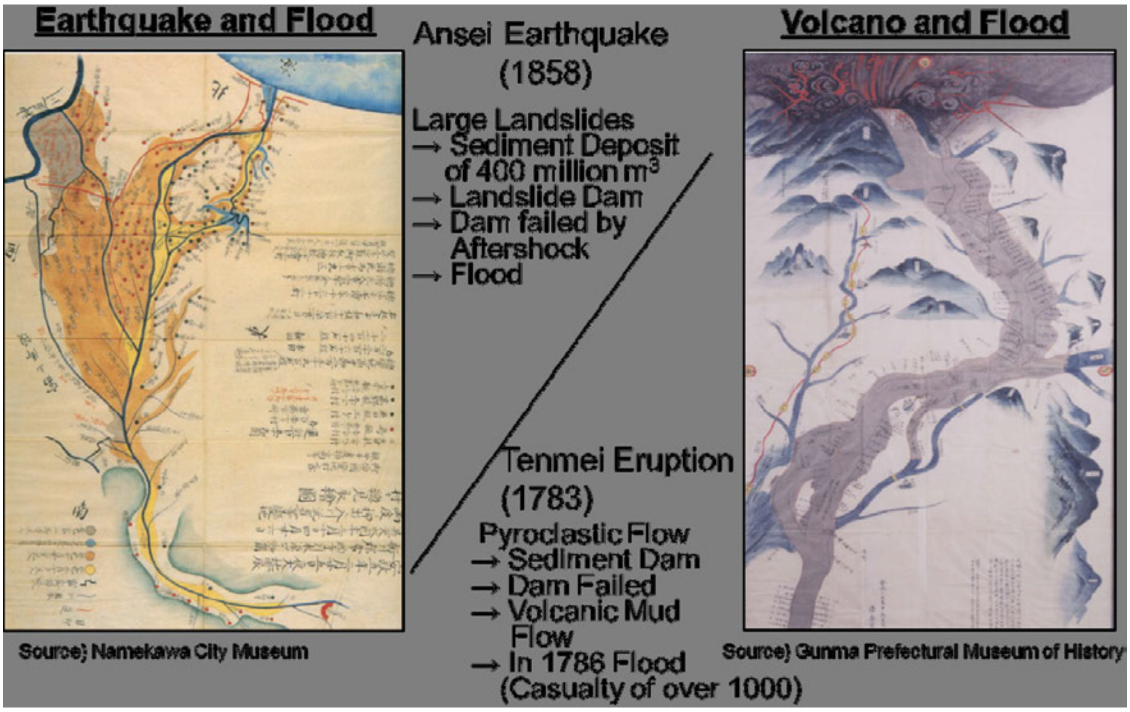

Fig. 4 Historical Complex Disasters in Japan

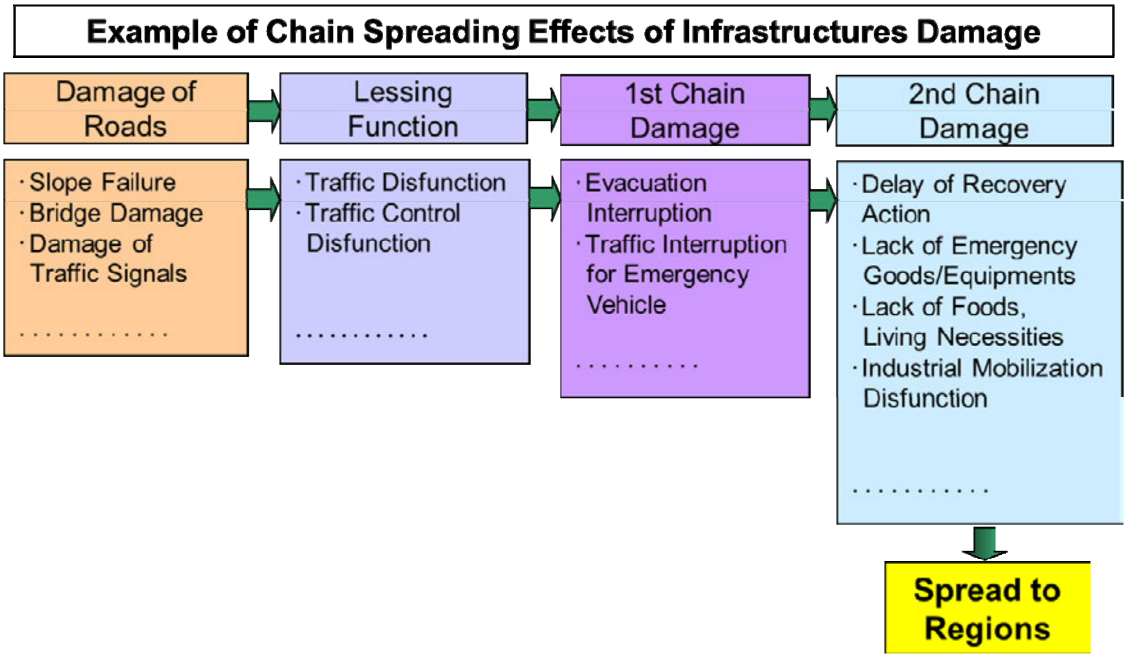

Fig. 5 Select out of Impact Spreading to be considered in development of Disaster Scenarios

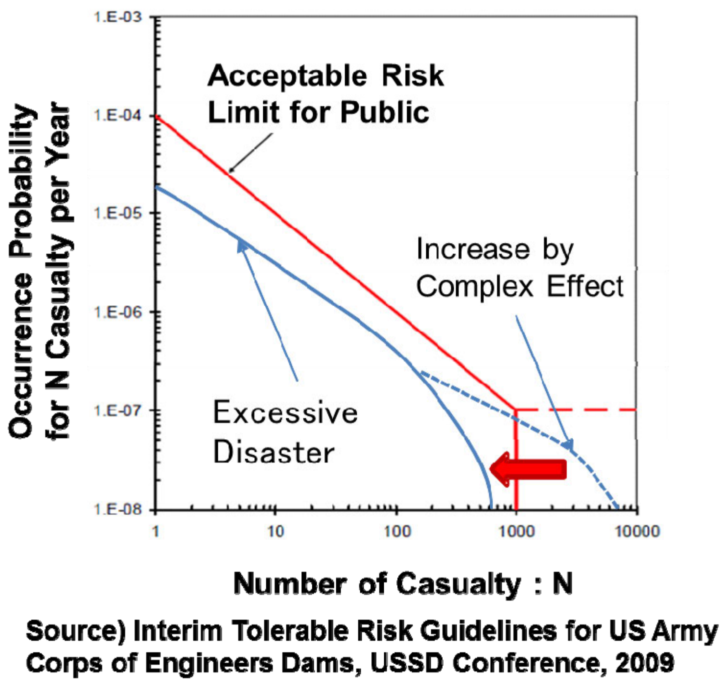

Fig. 6 Example of Evaluation Index of Disaster Risk and Impacts

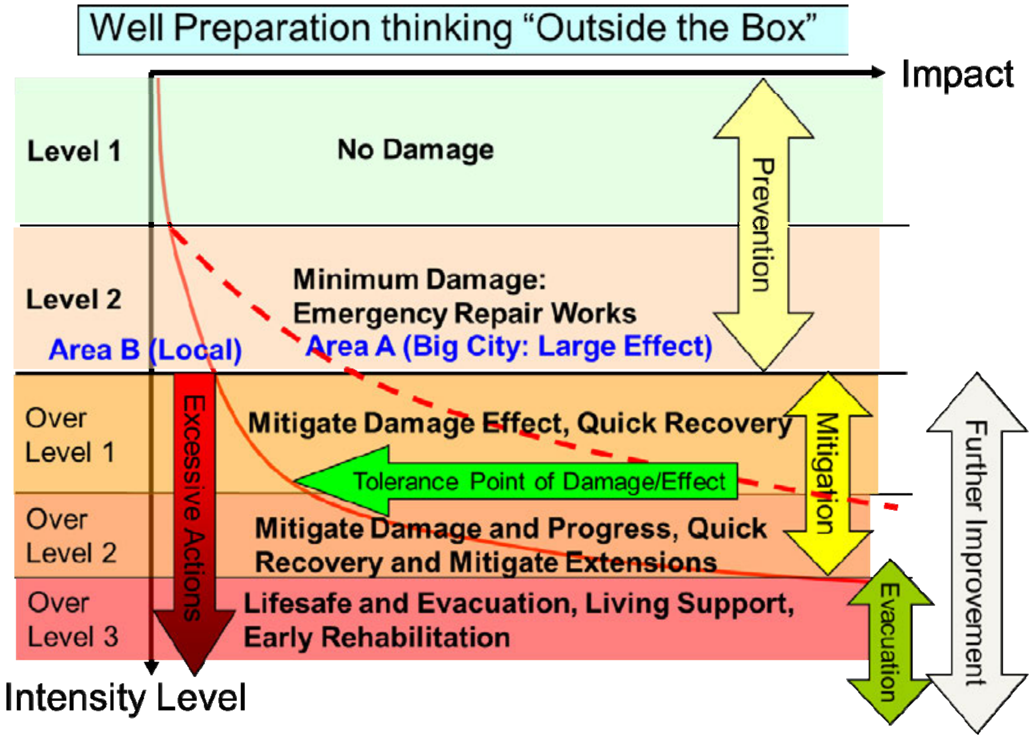

Fig. 7 Multi-level Disaster Response Concept dependent on Level of Disasters

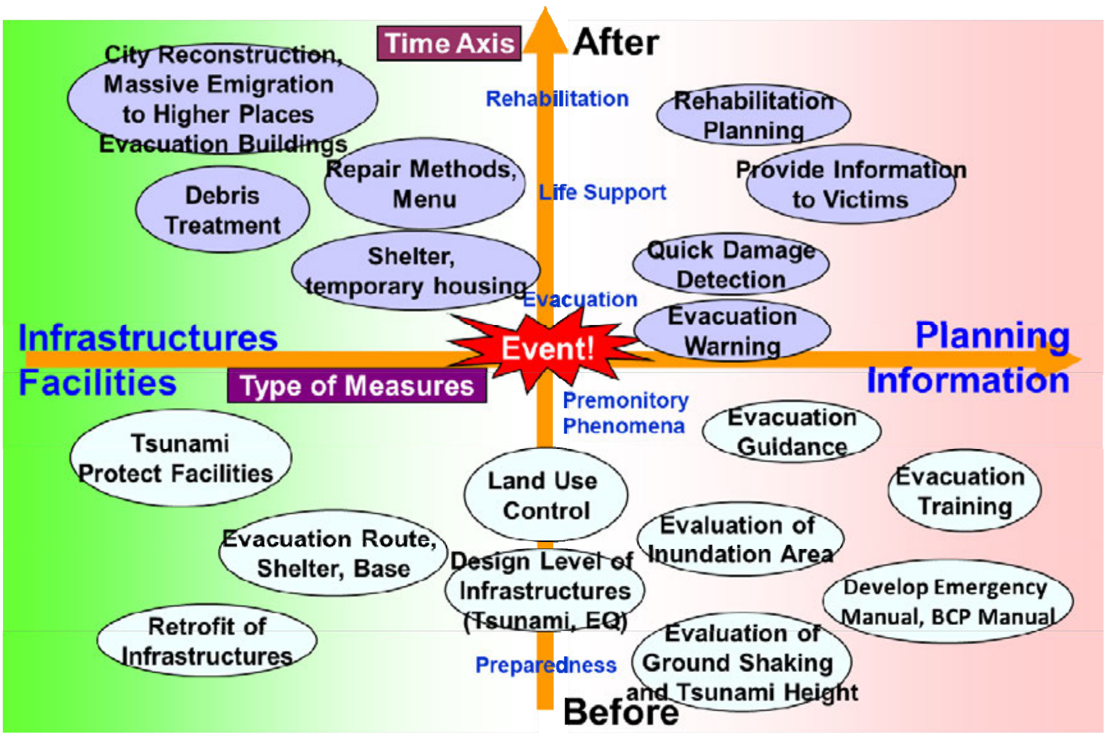

Fig. 8 Example of Countermeasures: Every Possible Effort based on "Mitigation/Evacuation"

Assessment of Design Long-period Earthquake Motions for High-rise and Base-isolated Buildings

by

Izuru Okawa[1]

ABSTRACT

It was presented in the previous joint panel in 2011 that we have made a proposal on method for evaluating the long-period earthquake motion time history with periods from 0.1 to 10 second using designated earthquake magnitude, shortest distance to the source area and location of hypocenter in view of establishing the generation scheme for design long-period motions for super high-rise buildings and applied it for the simulation of large subduction-zone earthquakes. The 2011 Off the Pacific coast of Tohoku earthquake has provided us with an opportunity to examine the method by comparing recorded motions with simulated ones. We had made revisions to this proposed method. We also made simulations for the Tohoku earthquake with the revised method. We have obtained better fit of the formula to the recorded motions. In addition, long-period ground motions were simulated for a three-events-connected source model expected to occur in the Nankai trough region.

KEYWORDS: Attenuation, Group delay time, Long-period ground motion, Site coefficient, the 2011 Off the Pacific coast of Tohoku earthquake

1. INTRODUCTION

A large earthquake is supposed to occur on subduction zones around Japan in near future. We have serious concerns on structural damage due to the long-period ground motions generated by the earthquake. We have developed a method to evaluate the long-period ground motion both in spectral and time-history formats (Satoh, 2010a), and are going to apply the method for constructing the input ground motion for the design of high-rise buildings and base-isolated buildings, concurrently performing earthquake response history analysis with the simulated motions to confirm the influences of those motions to building structures. The problem for the method was insufficiency of recorded motions especially for larger events.

During the 2011 off the Pacific coast of Tohoku Earthquake (hereinafter referred to as the Tohoku earthquake), the wide area in Japan suffered extremely large earthquake motions, the earthquake data for larger events including many aftershocks became available. It also caused a situation whether the proposed method can be applied to large event or not. During the earthquake, the high-rise buildings in large cities shook largely and some disorders with non-structural members were reported. For the expected huge earthquake in future, the long-period motions lasting long will be generated, and we confirmed that the effect on buildings with longer natural periods such as super high-rise buildings will be significant.

In this report, we first revised our previously proposed formula for generating the long-period motions with large subduction-zone earthquakes, using the added recorded motions including records from the Tohoku earthquake. We have successfully revised the method. After confirming the better fit of the revised method to the recorded motions for larger earthquake, we additionally made some simulations for future large events. The advantage of the method we have proposed can be utilized easily to generate a long-period time history once the earthquake magnitude and hypocentre are fixed with small number of necessary parameters, and it will surely be an excellent reference values for judging the validity of the motions simulated with other various methods.

In this paper, for enhancing the applicability of our proposed method when it is applied for

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extremely large events, the formula for Sa with 5% damping is considered to include a first- and second-order polynomial, i.e., M_w and M_w^2. In addition, for both of spectrum and group delay time, the distance dependency is separately considered for events on either of the Pacific and the Philippine plates. The site factors for amplification and group delay time are also separately considered for the two plate events, in case the thickness of sediment under recording station above seismic bedrock is large enough with the travelling time of seismic wave of longer than 1 second. Although similar empirical formulas for the long-period response spectra involving site response factors have been proposed by other researchers, the considerations of the second-order term M_w^2 and the difference between the plates have not been applied yet. (Satoh et.al., 2012a)

2. Evaluation of Long-period Ground Motion with Subduction-Zone Mega-Earthquake Based on the Empirical Method

The research on the evaluation of long-period motions has widely been conducted using theoretical method such as the 3D-FDM. On the other hand, the researches with the empirical evaluation of the long-period motions are very few. (Kataoka, 2008) showed the attenuation formula for evaluating the response spectral properties. However, almost no research has targeted on the time history generation.

Considering the usefulness of the formula and expecting the data accumulation in future, the empirical method will become much more useful in engineering sense. In addition, the evaluated motion with the empirical method will be useful enough to judge the plausibility of the theoretical method.

We used nationwide many ground motion records to make an empirical model to predict the ground motion with 0.1 to 10 second period range. Furthermore, based on this formula, we investigated and proposed the method to construct the long-period ground motion time histories generated by hypothetical large future earthquakes. (Satoh, et.al., 2010a, Okawa, et.al., 2010)

We considered the problems to be solved as follows to make revision on our previous proposal. One point is that the total moment magnitude M_w of the Tohoku earthquake was large and each of the sub-events, even when being decomposed, was still larger than the database range in magnitude. Therefore, the evaluation is still an extrapolation, and in addition, the logarithm of response spectral value is related with only M_w term, then, the value is resulted in excess for larger magnitude. When the source spectrum is represented with the ω^{-2} model, the amplitude level in longer period will surely be overestimated.

2.1 Additional Data for Revision of Attenuation Formula

For modification of the formula, the recorded data for 19 subduction-zone earthquakes occurring during the period that followed 33 subduction-zone earthquakes from 1988 to July, 2007 that was used for making the previous version of formulas. The totally 52 earthquake epicenters were plotted in Fig.1 (a). The 2009 Suruga Bay earthquake is from the Philippine plate among the added data. The off south-west Hokkaido earthquake occurring in the eastern fringe of Japan Sea was included in the group of the Pacific plate earthquake. The 2011.3.9 foreshock of the Tohoku earthquake and the 2011.3.11 off Iwate prefecture aftershock at 15:08 were characterized with moment magnitude M_0 and fault plain analyzed by JMA. For the 2009 Suruga Bay earthquake, the M_0 and fault plain analyzed by Suzuki & Aoi was used. For other events, M_0 from F-net operated by NIED was used and the JMA hypocenters were used as point source. The Tohoku earthquake was not included in the database, since the earthquake should be modeled as a connected-sources event consisting several strong motion generation areas representing the earthquakes from the empirical formula representing the period range between 0.1 to 10 second, however, the regression coefficients vary due to the selections of source model of the Mw=9 event that is not determined yet.

The data used here are from K-NET, KiK-net, JMA87 and JMA95 records for events prior to the 2009 Suruga Bay earthquake for Kanto, Nohbi and Osaka plains and the records from the 1st floor sensor at Kogakuin University in Shinjuku, Tokyo.

The selection criteria of Subduction-zone event records are as follows,

1) Subduction Type :Mj>6.5 for hypocentral distance<400km (Mj:JMA magnitude)
2) Recording station within the distance for which the PGA is equal to or greater than 2 cm/s^2 with the Fukushima-Tanaka attenuation formula (Fukushima et.al., 1992)
3) The motion is recorded from the S-wave arrival and reliable for period 0.1 to 10 second.

The recording stations are plotted in Fig.1 (b). The scatters of data for shortest distance to the fault and the moment magnitude are plotted in Fig.1(c). The smallest value for the shortest distance is 20 km. The largest M_w is 8.2 for 1994 Eastern Hokkaido earthquake and 2003 Tokachi-oki earthquake.

The analytical scheme is same as the previous study (Satoh, et.al., 2010a). The response spectra and the group delay times are evaluated from the data portions after the S-wave arrivals. The response spectrum stands for the geometrical average of the spectral values for two horizontal components and the mean and variance values for group delay time stands for those for the arithmetic mean values for two horizontal components. In addition, the mean and the variance values of the group delay time are calculated from time history data with time interval 0.02 second and total duration time of 1310.72 sec. with zero padding if necessary. The bandwidth for which the mean and the variance are computed is 0.049 Hz.

2.2 Revision of Attenuation Formula for Acc. Response Spectra with 5% damping in Longer Period

In the least square analysis, the 5% damping acceleration response spectra $S_a(T)$ is related with the moment magnitude and the shortest distance from recording station to the assigned source area of each recorded event. Our initial formula was Eqn. 2.1. (Satoh, et.al., 2010a) The second formula Eqn. 2.2 was used for revision with classified data. (Satoh, et.al., 2012a)

$$\log_{10} S_a(T) = a(T)M_w + b(T)R - \log_{10}(R^{p(T)} + d(T)10^{0.5M_w}) + c_0(T) + c_j(T) \quad (2.1)$$

$$\begin{aligned}\log_{10} S_a(T) = {} & a_1(T)M_w + a_2(T)M_w{}^2 \\ & + be(T)R + bw(T)R \\ & - \log_{10}(R^{p(T)} + d(T)10^{0.5M_w}) + c_0(T) \\ & + c_j(T) + c_{wj}(T)\end{aligned} \quad (2.2)$$

Where, T is period in second, and 55 values are selected for evaluation from 0.1 to 10 second. The M_w is the moment magnitude and R is the shortest distance from the recording site to the source area, and $a(T)$, $a_1(T)$, $a_2(T)$, $b(T)$, $be(T)$, $bw(T)$, $p(T)$, $d(T)$, $c_0(T)$, $c_j(T)$, $c_{wj}(T)$ are coefficients to be determined with the least squares analysis. $a_1(T)$, $a_2(T)$ are coefficients representing the source properties. $be(T)$, $bw(T)$ represent the property for propagation from the Pacific plate and the Philippine plate, respectively, and either coefficient is selected with the location of the event source. The coefficient $c_0(T)$ is assumed to be the site amplification factor for KiK-net FKSH19 station which is regarded as benchmark station on the seismic bedrock and, $c_j(T)$, $c_{wj}(T)$ are site coefficients for the j-th recoding station. $c_j(T)$ is used basically for each station. However, when recording site is on the Kanto plain, and event is from the Philippine plate and the seismic wave travelling time (Tz3.2) from the bedrock (the shear wave velocity Vs=3.2km/s) to the upper engineering base layer at site is greater than 1 sec., the coefficient $c_{wj}(T)$ is used instead of $c_j(T)$. The values for $10^{c_j(T)}$, $10^{c_{wj}(T)}$ are site amplification factors when T is larger than 1. The Tz3.2 can be evaluated using the deep sediment structure data disclosed by the Headquarters for Earthquake Research Promotion (HERP), MEXT. The site coefficients

$c_j(T)$, $c_{wj}(T)$ are actually evaluated as the weighted average of the coefficients for the crustal earthquakes and Subduction-zone earthquakes, since the number of subduction earthquakes are very small.

The coefficients for formula Eqn. 2.2 are shown in Fig.2(a). The two plots for coefficient b means that the red fine broken line is for be and blue dotted line is for bw. The plot e, additionally, indicates the regression error, i.e., corresponding to the standard deviation of logarithmic differences between recorded and predicted values. The following chapters refer the terms, mean (μ) and mean +standard deviation ($\mu+\sigma$) levels. The standard deviation corresponds to the regression error e that is shown here. The difference between be and bw indicates that the attenuation rate for distance differs for the two seismic source areas.

The site amplification coefficients are shown in Fig.3 for Tokyo, Nagoya and Osaka area. The contours are estimated from the values for recording stations.

2.3 Revision of Empirical Formula for Frequency-dependent Mean and Variance of Narrow-band Group Delay Time

The mean value μ_{tgr} of the group delay time corresponds to the gravity center of arriving time of wave group in a narrowband. The standard deviation σ_{tgr} of the group delay time corresponds to the scatter of the arriving time that is the duration time of the wave group in the narrowband. Since the group delay time is the first derivative of the Fourier phase spectra, once the initial phase angle is fixed, the other phase angles are calculated recursively, assuming a normal distribution with the mean and standard deviation values within the narrowband. The method holds an advantage to realize the spectral non-stationarity of the wave seemingly caused by the dispersion of surface waves. The average values are corrected so that the rupture initiation time should be zero.

Since both of the average group delay time μ_{tgr}, and the variance σ^2_{tgr} of group delay time can be related with the source property, the path effect and the site characteristic, both μ_{tgr} and σ^2_{tgr} were eventually related with such parameters. We have evaluated both properties with the previous study. As was done for the spectral formula, we have also revised the formula considering the wave propagation from sources and site specific effect on the group delay times with the following formula.

$$\begin{aligned}
\mu_{tgr}(f) &= A_{1tgr}(f)M_0^{1/3} + Be_1(f)X + Bw_1(f)X \\
&\quad + C_{1j}(f) + Cw_{1j}(f) \\
\sigma^2_{tgr}(f) &= A_{2tgr}(f)M_0^{1/3} + Be_2(f)X + Bw_2(f)X \\
&\quad + C_{2j}(f) + Cw_{2j}(f)
\end{aligned} \tag{2.3}$$

Where, M_0 is seismic moment in dyne-cm, X is the hypo-central distance in kilometer, f is frequency. A_{1tgr} , $Be_1(f)$, $Bw_1(f)$, $C_{1j}(f)$, $Cw_{1j}(f)$, A_{2tgr} , $Be_2(f)$, $Bw_2(f)$, $C_{2j}(f)$, $Cw_{2j}(f)$ are determined by the least squares analysis. $C_{1j}(f)$, $Cw_{1j}(f)$, $C_{2j}(f)$, $Cw_{2j}(f)$ are called site coefficients. These coefficients are shown in Fig.2(b). The coefficient for distance dependence is shown with 1/B. The Fig.2(b) left shows for μ_{tgr}, and the Fig.2(b) right shows for σ^2_{tgr}. The coefficents E that are presented in both cases indicate the regression errors in the similar manner for response spectrum formula. The value 1/B indicates the propagation velocity of seismic waves radiated from the seismic source. The values differ between cases for the Pacific plate and the Philippine plate. The revised formula indicates that the duration time is longer than the previous formula. There are still insufficient data with sufficient recording time. With this revision, the new data were added. These added data generally hold longer recording time. The aforementioned note will be owing to such situation with the current earthquake observations.

3. VALIDATION OF PROPOSED EMPIRICAL FORMULA WITH RECORDED MOTIONS

The empirical formulas for response spectrum and group delay time (tgr) were applied to the Tohoku earthquake, its foreshock and the largest aftershock. The moment magnitude and macroscopic fault model for the foreshock of March 9th were based on the JMA analysis (JMA, 2011a,b). However, the fault model for the largest aftershock at March 11th, at 15:15 JST was not announced by JMA yet. Therefore, its moment magnitude was taken the value from F-net operated by NIED, and the dip angle and rupture direction of the fault was assumed based on the Harvard CMT solution. The fault length, width and its area were calculated assuming static stress drop of 3MPa with square fault. In addition, the JMA hypocenter was placed at the middle of the fault plain as adopted as the point that initiates rupture. The values for the static stress drop 3MPa is close to the average for the world historical shallow earthquake faults database and even for the earthquakes occurring on the plate boundary. The value, 3Mpa is therefore used for source model with the three connected-earthquake for Tokai-Tonankai-Nankai established by the Central Disaster Management Council (CDMC), Cabinet Office (CAO, 2012).
Furthermore, the source model for the main shock, the Tohoku earthquake, was set up based on the model by Satoh that utilized the empirical Green's function and the recipe for strong motion prediction by the HERP, MEXT (Satoh, 2012b). The moment magnitudes for three faults 1, 2, and 3 are, 8.4, 8.8, and 8.2, respectively. The three macroscopic fault plains and four sets of strong motion generation area on fault and the hypocenters are presented in Fig.4. In the figure, the distributions of the site amplification factors for 3 second period are also shown.

4. Simulation of three-connected earthquake motions in Nankai trough

The long-period motions due to the mega-earthquake along the Nankai trough were simulated. The huge earthquake is assumed as three-connected earthquake for Nankai, Tonankai and Tokai earthquakes. The rectangular source model (Tsurugi, 2005) was used for Nankai and Tonankai earthquakes following CDMC (CDMC, 2003). We set up a rectangular source model for Tokai earthquake employing the HERP model used for the long-period earthquake motion map (HERP, 2009).
The method presented here generates each simulated motion as a sample of random process. The rupture sequences of faults also follow the assumption by CDMC as indicated in Fig. 5 with red arrows. There are variations among sample waves depending on the random numbers chosen for each simulation. The rectangular source shapes and locations are shown in Fig.5, together with the locations of the stations for which the long-period motions were generated. The moment magnitudes of sub-sources were 8.2 (Nankai-east) 8.4 (Nankai-west), 7.9 (Tonankai-east), 8.0 (Tonankai-west) and 8.0 (Tokai), and total moment magnitude amounts 8.7. The 21 sample motions were generated for ground surface at each station and one sample motion that is the closest to the average of them was taken and the pseudo velocity response spectra were computed.
The pSv spectral values were plotted in Fig. 6 for 4 recording station, named as OSKH02 (KiK-net) for Konohana on the Osaka bay, AIC003 (K-NET) for Tsushima, Aichi, SZO024 (K-NET) for Hamamatsu, Shizuoka, and KGIN for Kogakuin Univ. campus in Shinjuku, Tokyo. The blue solid line indicates the spectrum corresponds to the nearest one with the mean spectra in 21 simulated motions with different random numbers, the blue dotted line indicates that the spectra corresponds to the mean + standard deviation level spectra that was computed using the regression errors.

During the 2011 Tohoku earthquake, there are no recording sites where the pSv spectra with 5% damping above 5 second of period exceeded 80 cm/s, the level of the BSL notification at engineering bedrock. However, we have many recording sites exceeding the value for the periods less than 5 second. The K-NET site MYG006 in Miyagi prefecture exceeded the level 100 cm/s for 5% pSv in broad period range, 0.1 to 4 second. The amplitude is larger than the simulated motions at SZO024 for the three-connected earthquake motions.

The CDMC has recently released the new seismic source model for huge earthquake occurring along the Nankai trough region that renews the previous source model for the future Nankai-Tonankai-Tokai earthquake (Mw=8.7) and is almost double in earthquake size (Mw=9.0). They have also reported the estimated seismic intensity and tsunami height for wide areas influenced by this earthquake.
We will be also working for evaluating the long-period motions with this new earthquake sources for wide areas applying our revised new formulas.

5. Summary of Computed Responses of High-rise and Base-isolated Building Models to Simulated Motions

The above-mentioned three-connected simulated motions for four representative sites for Tokyo, Nagoya, Osaka and Hamamatsu were applied for earthquake response analyses with various types of high-rise and base-isolated buildings.

5.1 high-rise building models
Six steel high-rise building models corresponding to the heights with 100 to 250 meters and the first mode natural periods, 2.3 to 6.5 seconds, seven RC high-rise building models corresponding to the heights with 90 to 240 meters and the first mode natural periods, 1.9 to 5.4 seconds were used for analyses. In addition, the input motions with two levels, i.e., mean (μ), mean+regression error ($\mu+\sigma$) were applied.
The structural design criteria for high-rise building with level-2 motion (collapse protection level) in Japan are basically stipulated that the maximum story drift ratio is within 1/100, and the story ductility ratio is less than 2.0. This is referred to as the criteria, hereafter.
The computation results were briefly summarized as follows in focus on the drift rations..

(1) For OSKH02 motion representing the Osaka bay area, responses of building models taller than 150 meters to mean (μ) motion exceeded the criteria. In addition, all models failed to satisfy the criteria to $\mu+\sigma$ motion. It also showed difficulties to satisfy the criteria even if some retrofit countermeasure works such as equipping with response control dampers are considered.
(2) For AIC003 motion representing Nagoya area, all building models satisfied the criteria for mean (μ) motions, all models satisfied the criteria, for $\mu+\sigma$ motion, models except for 150 meter height satisfied the criteria.
(3) For KGIN motion, representing the Shinjuku area, most of the building models shorter than 200 meters satisfied the criteria. However, some models taller than 200 meters showed values larger than the criteria.
(4) For SZO024 motion, representing Hamamatsu area located above the earthquake source area, larger story drift ratios exceeding the criteria were found for $\mu+\sigma$ motions for any heights of the buildings.

5.2 Base-isolated building models
The building models used here are from 22 existing buildings. They vary with construction age (1987-2008), height of superstructure (11.9-140 meters), Devices (LRB, NR, HDR, SD,LD, OD etc.). The natural periods of superstructures are 01-3.5 sec. and those for 200% strain level are 1.8-6.4 sec.
The computation results are briefly summarized as follows.

(1) For mean (μ) motions, most of the superstructures are in the state within the short term allowable limit.
(2) For ($\mu+\sigma$) motions, 5 to 8 percentile superstructures exceeded the bearing capacity for OSKH02 and SZO024 motions. Totally 15 percentile models exceeded the elastic limit. Deformations of devices exceeded the limits for models with longer natural periods for OSKH02 motions.

6. CONCLUSIONS

We have revised our proposed formula for evaluating the long-period earthquake motions using newly collected data with the 2011 Tohoku earthquakes and its aftershocks and have also successfully confirmed the validity of the formula. The long-period motion for the main shock of the Tohoku earthquake was simulated with three-connected earthquake model. We found that the application of the formula with some upper limit of magnitude was appropriate.

The attenuation properties were compared between earthquakes occurring on the Pacific plate and Philippine plate, and the difference was confirmed. The difference of site amplification and the site coefficients for properties on the group delay time was also large for Kanto area in case the predominant period between seismic bedrock and engineering bedrock (Tz3.2) is larger than 4.0 second. We have also confirmed that the site amplification is generally larger and the duration time of seismic motion is longer for the earthquakes in the Philippine plate than in the Pacific plate.

We applied the revised formula with parameters corresponding to the mean and the mean + standard deviation levels. For application to building design, we need to fix the parameter values appropriate for design considerations, i.e., the pertinent engineering parameters relevant to levels of earthquake motions, building responses and occurrence rate of these values.

As was mentioned previously, the new seismic source model for huge earthquakes occurring along the Philippine plate region was released from CDMC (2012, CAO). We are going to evaluate the long-period motions with these renewed conditions for wide areas applying our revised new formulas.

In addition to the simulations of long-period motions for large events, we have computed the structural responses of high-rise and base-isolated buildings with variety of building heights with steel and reinforced concrete structural models, applying the time histories generated with the developed methods. The computation results will be summarized and reported in detail after completing the analyses.

ACKNOWLEDGEMENT

The study presented in this report is based on the collaborative work conducted between BRI the Ohsaki Research Institute, the Japan Structural Consultants Association (JSCA) and the Japan Society of Seismic Association (JSSI). We used the strong motion records from K-NET, KiK-net, the earthquake mechanism solution of F-net from NIED, the JMA 87 and 95 type strong motion records, and earthquake information from JMA, and the data of the Harvard University CMT solutions. We also acknowledge the usage of strong motions records from the campus building for Prof. Hisada, Kogakuin University, Shinjuku, Tokyo. In addition, the GMT was used for making maps (Wessel, 1998). This work was conducted in the task topic, "Study on the effect of long-period earthquake motions to the super-high-rise building," in the 2011 Promotional Project for Upgrading the Building Standards under the auspices of the Ministry of Land, Infrastructure, Transport and Tourism. The authors express their sincere thanks to the members of the Committee for the Study of the Long-Period Earthquake Motions established in Ohsaki Research Institute, Inc. for their relevant ideas and fruitful discussions.

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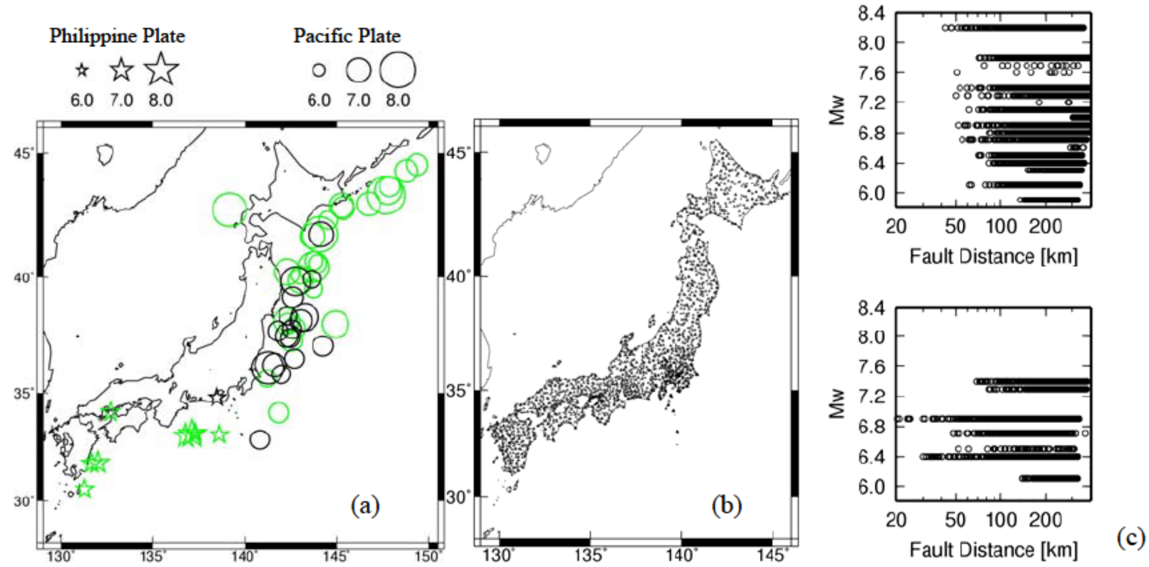

Figure 1. (a) Locations of epicenters, (b) Locations of recording stations, (c) Scatters of Mw, R of data

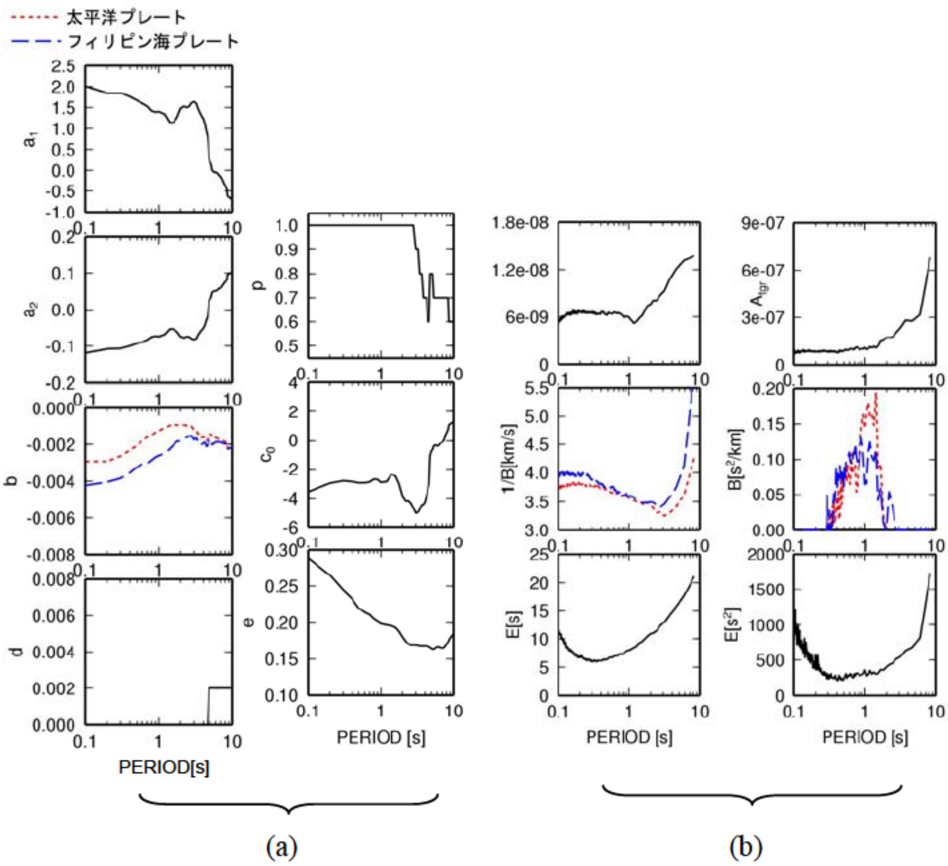

Figure 2, (a) Regression coefficients for formula Eqn. 2.2, (b) Regression coefficients for formula Eqn. 2.3, (b) left shows for μ_{tgr} and (b) right shows for σ^2_{tgr}

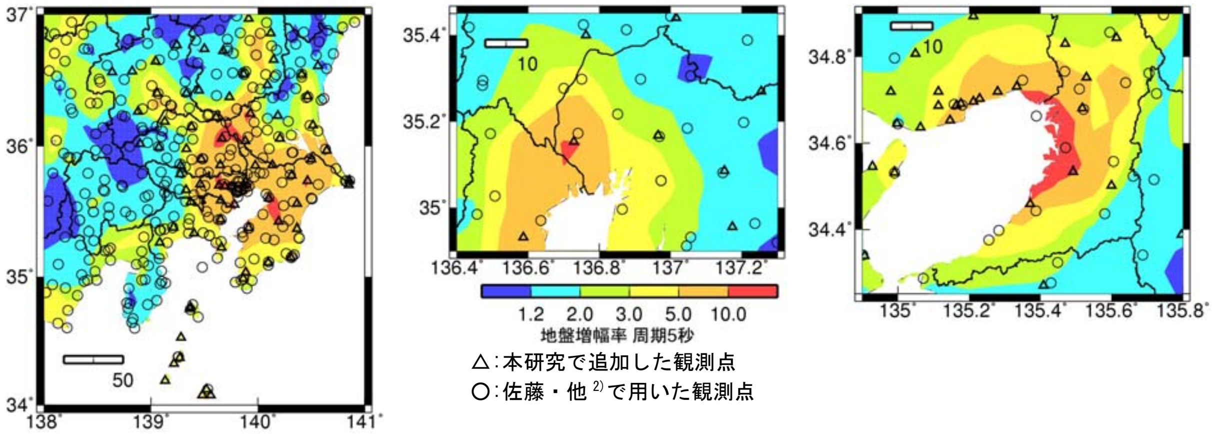

Figure 3. The distribution of site amplification coefficients $c_j(T)$ for three areas, i.e., Tokyo, Nagoya and Osaka. The coefficients are for period of 5 seconds. The circles indicate the recording stations used for this study, The triangles are added and also used in this study

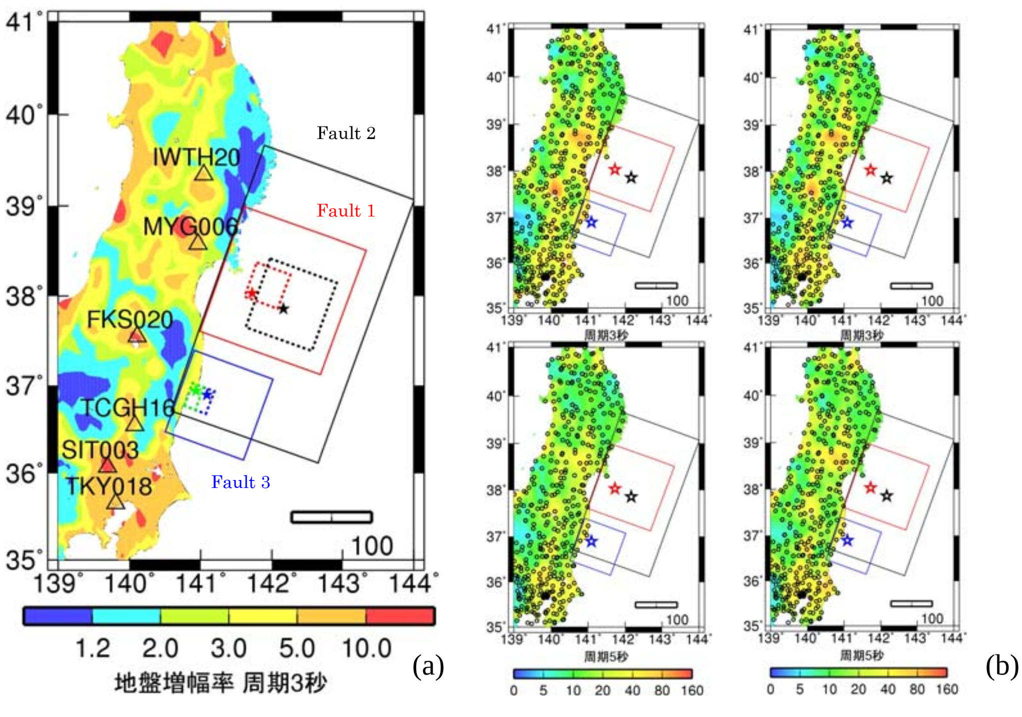

Figure 4. (a) The three macroscopic fault source models representing the 2011 Tohoku earthquake. The coloured contour shows the revised site amplification factor for Sa at 3 second of period. (b) left the pseudo velocity response spectra (pSv) at period 3 and 5 second with recorded motions for main shock, (b) right the pSv at 3 and 5 second for simulated motions with revised formula for main shock. (pSy are all with 5% damping)

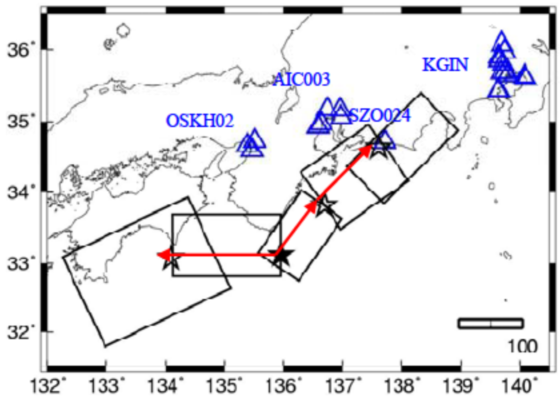

Figure 5. Three-connected earthquake model and the selected stations for evaluation of long-period motions.

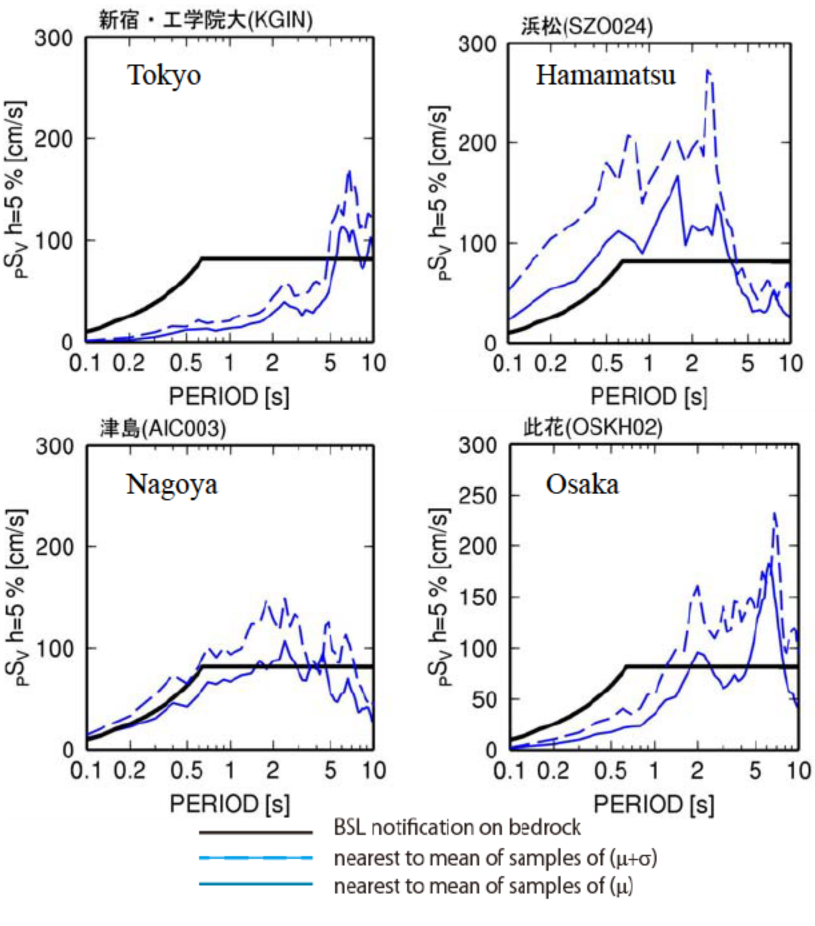

Figure 6. Pseudo velocity response spectra for simulated motions for three-connected earthquake for 4 stations

Brief Review of Building Damage by The 2011 Tohoku Japan Earthquake and Following Activities for Disaster Mitigation

by

Masanori Iiba[1], Isao Nishiyama[2], Hiroshi Fukuyama[3], Izuru Okawa[1] and Yasuo Okuda[4]

ABSTRACT

The 2011 Tohoku Japan earthquake generated large ground motion and gigantic tsunami in Tohoku and Kanto areas of the northeastern part of Japan. Since the hypocentral region is widely located off the coast of Japan, damages of many buildings and residential land by earthquake motions and devastating damages by tsunami attack occurred. The brief review of the building damages is presented, based on the research and reconnaissance reports of the Building Research Institute (BRI) and the National Institute for Land & Infrastructure Management (NILIM). Research Acitivities according to damages and seismic behaviors of buildings for disaster mitigation, such as prediction of long-duration and long-period earthquake ground motion for design use, problems of ceilings, liquefaction countermeasure for residential houses, and countermeasure for tsunami force are introduced.

KEYWORDS: 2011 Tohoku Earthquake, Building Behavior, Building Damage, Research Action, Disaster Mitigation

1. INTRODUCTION

The 2011 Tohoku Japan earthquake of moment magnitude (*Mw*) 9.0 occurred at 14:46 JST on March 11, 2011 and generated large ground motion and gigantic tsunami in Tohoku and Kanto areas of the northeastern part of Japan. This earthquake occurred at the boundary between the North American and Pacific plates resulted in people death of 19,213 (including missing) and totally collapsed houses of 128,525 as of 27 January 2012. The hypocentral region with approximately 450km in length in the NS direction and 150km in width in the EW direction gave the seismic intensity of 6- or more according to the Japan Meteorological Agency (JMA). As a result, damages of many buildings and residential land by earthquake motions and devastating damages by tsunami attack occurred in the coast lines of Tohoku and Kanto areas.

The BRI and the NILIM sent 43 teams in total for field survey and summarized three damage reports [1, 2, 3]. So as to reflect lesson learnt from the earthquake to practices such as the revision of building structural codes, the BRI and the NILIM are collaboratively carrying out coping activities on picked up issues with the help of the administration by the Ministry of Land, Infrastructure, Transport and Tourism (MLIT).

In the paper, brief review of the building damage is presented which is based on the research and reconnaissance reports. Several key issues to be coped with by the building structural codes are identified. They are 1) prediction of long-duration and long-period earthquake ground motion for design use, 2) problems of fallen down of ceilings and so on, 3) liquefaction countermeasure for residential houses, 4) evaluation of tsunami force, etc. Then, the state of the on-going coping activities on each issue is introduced at the moment of a year and 10 months after the earthquake.

2. RECORDED GROUND AND BUILDING EARTHQUAKE MOTIONS

2.1 Strong Motion Network of Building by BRI

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The strong motion network (BRI Strong Motion Network Website [4]) established in 1957 covers currently more than 70 buildings in major cities across Japan. When the Tohoku Japan earthquake occurred, 58 strong motion instruments started up from Hokkaido to Kansai areas. Among them, 31 buildings including three seismically isolated buildings suffered a shaking with the JMA seismic intensity of 5- or more. After the earthquake, free access to all the recorded digital data is strongly requested even from oversea researchers.

2.2 Strong Motion Record of Damaged Building

At least 4 buildings suffered severe earthquake motions with some damage. One example of the damaged buildings is a 9 storied steel reinforced concrete building in Sendai city. This building has a long history of recording of strong motions. Among them, strong motion records on the ninth floor that were obtained during the 1978 Miyagi-Ken-Oki earthquake are well known to have exceeded a maximum acceleration of more than 1000cm/s^2. By that earthquake, multi-story shear walls suffered shear crack and later repaired to behave in a ductile manner. In the meantime, during the Tohoku Japan earthquake, the repaired multi-story shear walls suffered flexural failure. Figure 1 shows the records of the strong motions and the fundamental natural periods of the building (T) calculated every 10 seconds (Kashima and Kitagawa [5]. The T increased from 0.6 seconds to 1.5 seconds during the earthquake. The change of T clearly shows and is consistent with the building damage.

2.3 Long-period Earthquake Ground Motion

During the Tohoku Japan earthquake, long-duration and long-period earthquake ground motions were observed in Tokyo, Osaka and other large cities. One example is the 52+3 storied steel office building on the coast of Osaka Bay that is 770 km away from the hypocenter. Figure 2 shows the records of the absolute and relative displacement waveforms. The absolute displacements in the SW-NE and in the NW-SE directions on the 1st floor was less than 10 cm, but the 52nd floor in the building suffered a large motion with a zero-to-peak amplitude of more than 130 cm, which is thought to be due to a resonance phenomenon. This indicates the importance of the prediction of long-duration and long-period ground motions by mega-earthquakes possibly to occur in Nankai trough.

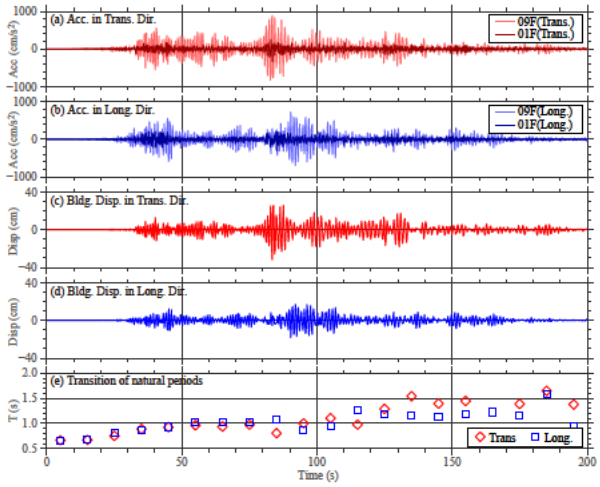


Fig. 1 Recorded strong motion and the calculated fundamental natural period

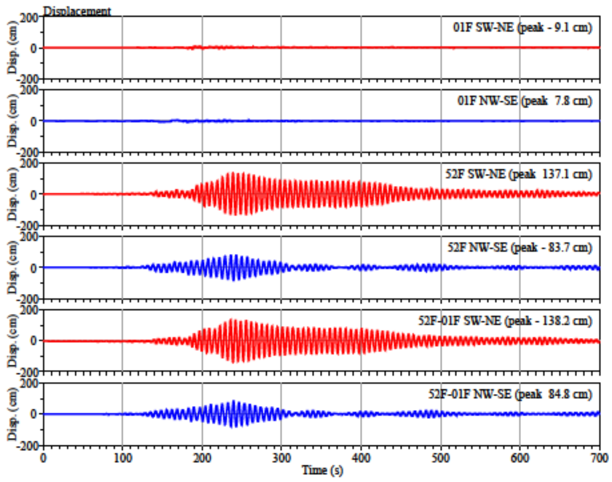


Fig. 2 Recorded strong motion of 52+3 storied steel office building located at 770km from the epicenter

3. BUILDING AND RESIDENTIAL LAND DAMAGE

The Tohoku Japan earthquake brought about building damage in a wide area of various prefectures on the Pacific coast in eastern Japan such as Iwate, Miyagi, Fukushima, Ibaraki and Chiba, and also brought about heavy liquefaction at the catchment basin area of Tone River and the reclaimed ground on Tokyo Bay, thus the BRI

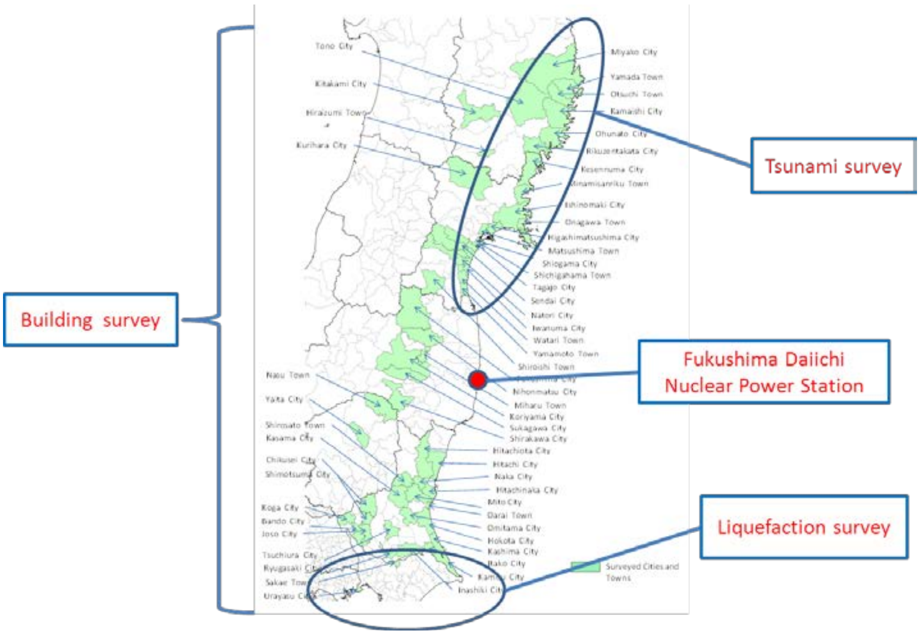


Fig. 3 Locations of field surveyed cities and towns by the BRI and the NILIM

and the NILIM selected the locations of the reconnaissance study (field survey) as shown in Fig. 3 with the exception of the area near the Fukushima Daiichi Nuclear Power Station. The field surveyed results are detailed in the reports [1, 2, 3].

3.1 Building Damages by Earthquake Motion

1) Wood houses: Most of the patterns of the damages to the wood houses were observed in past destructive earthquakes.

2) Steel buildings: Steel gymnasiums are surveyed extensively in Ibaraki prefecture, as the structural system of them is similar to that of factories and warehouses which are hard to be surveyed as they are private property. Most of the patterns of the damages were observed in past earthquakes, while the spalling of concrete at the joint of the steel roof structure and the reinforced concrete column shown in Fig. 4 and the fallen down of ceiling shown in Fig. 5 were marked.

3) Reinforced concrete buildings: Most of the patterns of the damages to reinforced concrete buildings were observed in past destructive earthquakes. So-called emergency operation buildings like city offices survived, but were not operational as shown in Fig. 6, which implies the necessity of higher level of performance in such buildings. Damage to the nonstructural walls adjacent to the door of residential buildings shown in Fig. 7 causes the similar problem. The retrofitted buildings behaved well in general with some exception.

Fig. 4 Spalling of concrete

Fig. 5 Fallen down of ceiling

Fig. 6 Survived but not functional

Fig. 7 Nonstructural wall failure

4) Seismically isolated buildings: Sixteen seismically isolated buildings in Miyagi prefecture and one in Yamagata prefecture were surveyed in which three buildings were instrumented and recorded strong motions. All of these buildings performed structurally very well and the steel dampers absorbed earthquake energy by the plastic deformation. However, the lead dampers suffered cracks due to many cycles of small amplitude of reversed deformation as shown in Fig. 8. Damage to the expansion joint was also seen quite frequently, which can be improved as early as possible.

Fig. 8 Lead damper cracked

5) Residential land - In the catchment area of Tone River and the coastal zone of Tokyo Bay, extensive damage such as sand boiling or ground transformation associated with liquefaction was confirmed. Highly tilted residential houses were seen, but visual cracks on the foundations were not observed as shown in Fig. 9. In Sendai city, the ground transformation by sliding of the housing site embankment was observed just like the one after the 1978 Miyagi-Ken-Oki earthquake.

Fig. 9 Tilted house by liquefaction

3.2 Building Damages by Tsunami

The coastal area along Aomori prefecture to Miyagi prefecture shown in Fig. 3, where northern part is ria coast and southern one is coastal plain, was surveyed. First, the building damage by tsunami was classified into several damage patterns. Next, about 100 buildings are carefully selected and studied in details such as on the dimension of the structure of the building, the maximum inundation depth at the building from the tsunami traces, damages of the building and so on, which were used in the study on tsunami evacuation buildings. Damage patterns by tsunami were classified as follows; 1) complete washed away (as shown in Fig. 10), 2) overturning with the effect of buoyancy (Fig. 11), 3) tilting after scouring (Fig. 12), 4) damage by debris impact (Fig. 13), and 5) survived from tsunami by shading effect of front buildings (Fig. 14).

Fig. 10 Complete washed away

Fig. 11 Overturning

Fig. 12 Tilting by scouring

Fig. 13 Debris impact

Fig. 14 Shading effect by front building

4. RESEARCH ACTIVITIES FOR COUNTER-MEASURE AGAINST DAMAGES

4.1 Coping Activities on Selected Issues

From the study and the survey explained above, the following issues were picked up. The coping activities, with the help of the MLIT, had been started and are still underway by the BRI and the NILIM. The research results are planned to be reflected to the revision of the building structural codes, which will be proposed by the NILIM after taking into account the expert opinions by the Building Structural Codes Committee as shown in Fig. 15.

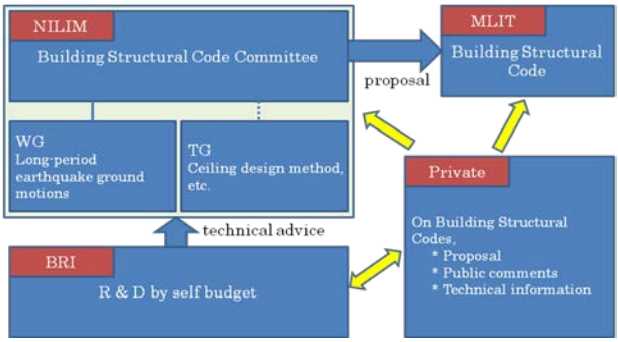


Fig. 15 Building structural codes committee in NILIM

- Possibility of free access to the digital data recorded by the BRI strong motion network (is under consideration in the BRI, consulting with owners of the instrumented buildings, etc.)
- Prediction of long-duration and long-period earthquake ground motion for design use, together with re-evaluation of structural performance under multiple cycles of loadings
- Addition to the building structural codes to deal with the problems of fallen down of ceilings and so on
- Evaluation of residual structural performance of fractured lead damper in seismically isolated buildings (was conducted by the Japan Society of Seismic Isolation)
- Liquefaction countermeasure for residential houses, for which neither structural calculation nor soil investigation is mandatory
- Evaluation of tsunami force necessary for the design of tsunami evacuation buildings

The building structural division of the MLIT supports the budget to promote the research and development for countermeasure projects and revision of the building structural code. The MLIT organized a production of promotion project for building-standards maintenance (promotion project). Many activities are going under the promotion production. When the new activity will be necessary, such as, the countermeasure against building damages, and accidents, a new item will constructed. The BRI contributes the setting theme and obtaining fruitful results with collaboration of the promotion projects

4.2 Long-duration and long-period Earthquake Ground Motion due to Next Earthquake

A social concern on the long-duration and long-period earthquake ground motions by mega-earthquakes at the subduction zone near ocean trench had been raised since the occurrence of oil tank fire during the 2003 Tokachi-Oki earthquake, and the prediction maps were announced based on detailed calculation (Headquarter for Earthquake Research Promotion Website [6, 7]).

The BRI and the NILIM with the collaboration of the MLIT adopted much practical empirical prediction method [8] based on the observations at about 1,600 recording stations across Japan. A proposal of empirical prediction had been released in December 2010 by the NILIM and the MLIT and received several hundreds of public comments. After the Tohoku Japan earthquake, numbers of high quality recorded data became available and additional validation studies on the empirical prediction method were carried out, taking into account the location of epicenters and the paths. Finally, revised evaluation method was proposed. Figure 16 shows the predicted velocity response spectra by the revised evaluation method for Nankai trough three-connected earthquake model. On August 29, 2012, it was announced officially the mega-earthquake model in Nankai trough by Cabinet Office [9], but the Headquarters for Earthquake Research Promotion has not yet completed the detailed calculation of expected long-duration and period ground motion as of November, 2012.

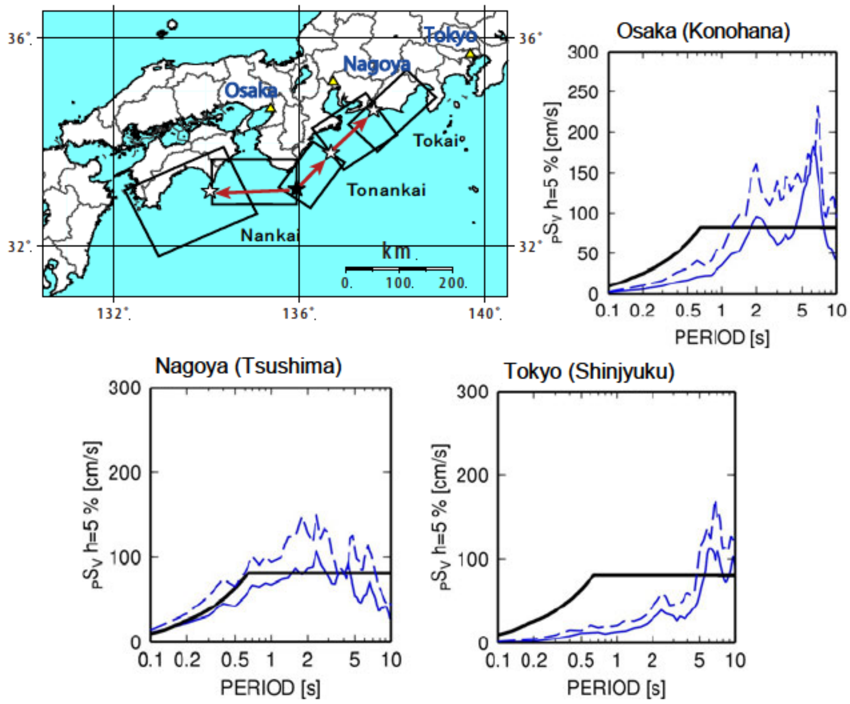


Fig. 16 Nankai trough three-connected earthquake model and predicted velocity response spectrum at Osaka, Nagoya and Tokyo

4.3 Behavior of Super High-rise and Seismically Isolated Buildings Under Long-period Earthquake Motion

The researches on evaluating the seismic behavior of super high-rise (reinforced concrete and steel) and seismically isolated buildings under the long-duration and long-period earthquake motions with the collaboration with the promotion projects by the MLIT. The researches on characteristics of members and isolation devices have been conducted under the experiment with multi-cycle loadings.

As for the reinforced concrete structure, in addition to statically repeated loading experiments of the columns, girders and their joints, the shaking table test on a 20-story frame model of 1/4 in scale and 20m in height was carried out, as shown in Fig.17. In the shaking table test, the story drift angle of the model reached a large deformation of 1/37rad. The behavior of the model was quite sable under the large deformation.

As for the steel structure, the static experiments for the columns, girders and their joint panels were conducted. Based on the plastic ratio and the accumulated plastic ratio, the fatigue damage potential of the steel members is evaluated. Dynamically multi-cycled tests of isolators and dampers were conducted, as shown in Fig.18. The dependency of the input energy on the characteristics of the devices, such as, heat generation, the changes of yield force or friction force was evaluated under the multi-cyclic excitation.

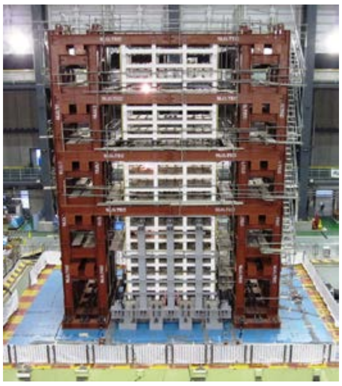


Fig. 17 Scale model of super high-rise RC building for shaking table test

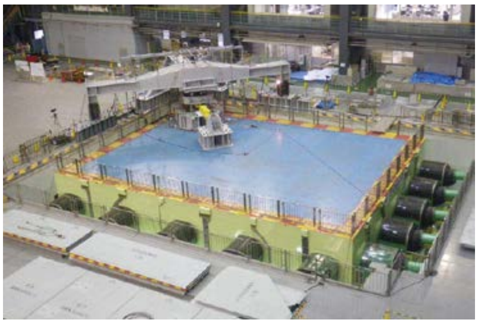


Fig. 18 Dynamically multi-cycled tests of isolators

4.4 Fallen Down of Ceilings and so on.

The problem of fallen down of the ceilings which cover large space such as gymnasiums has been indicated by the BRI and the NILIM since 2001 Geiyo earthquake. The technical advice to install appropriate amount of diagonal braces on hanging bolts and to keep appropriate clearance between ceiling and surrounding structure have been recommended from the MLIT, the BRI and the NILIM. In the Tohoku Japan earthquake, huge numbers of large space ceilings (about 2,000) fell down and even casualties occurred. Therefore, extensive detailed survey on the ceiling damages was conducted, where 151 damaged ceilings are collected and 11 of them were studied in detail. Based on this study in addition to the previous knowledge, the current qualitative technical advice is planned to be modified into much quantitative one. Fig. 19 compares the ceiling height and unit mass of the fallen ceiling with the cases of with and without injury.

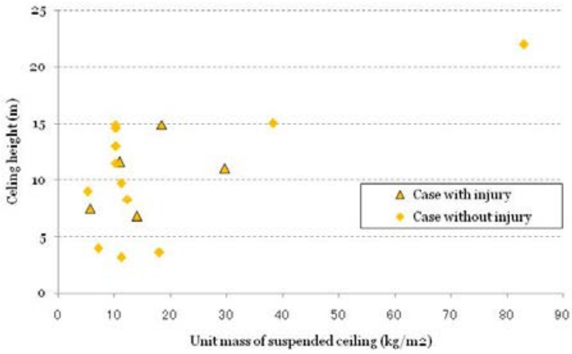


Fig. 19 Fallen ceilings with/without injury

After the earthquake, the NILIM and the BRI started to the countermeasure for improve the earthquake resistance of ceilings against the earthquake. The discussion items for improvement are specification for ceiling materials and setting, and the appropriate methods for calculating seismic response of ceiling and conditions for safety.

The fallen down of the escalator trusses in shopping centers were reported on October 26, 2011 by mass media. In ordinal practices, overlapping between the escalator truss and the girder on the upper story is selected as H/100+20mm, where H is the height of the escalator. Currently, the requirement of overlapped length is planned to be increased to H/40 with the exceptions with fall prevention device as shown in Fig. 20.

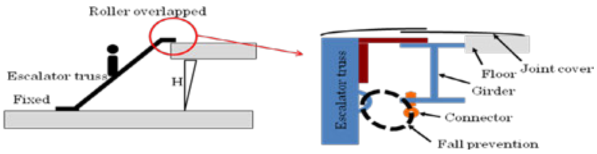
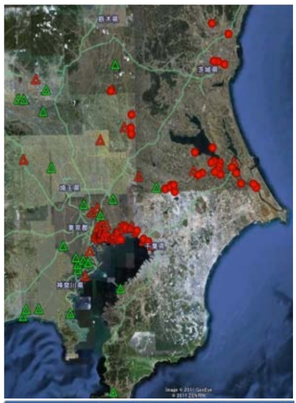


Fig. 20 Prevention of fallen down of escalator truss

4.5 Liquefaction Countermeasure for Residential Houses

For wood houses, the structural calculation is not mandated in the Japan's building structure codes. Thus, the liquefaction countermeasures is not clearly provided at present in the building construction for the detached houses.

The liquefaction evaluation by F_L-method was carried out in selected 112 sites in Kanto area [10], and the results were compared with the observation as shown in Fig. 21. All liquefied sites were predicted, but still many sites without liquefaction were cautioned. These require the further improvement of evaluation accuracy. So as to apply F_L-method, N-value by SPT (standard penetration test), fine fraction content, water level, and so on are needed. The cost necessary for getting the information is not affordable for the owner of residential house. The BRI is now trying to study the possibility of only using SWS test (Swedish weight sounding test) plus water level and soil judgment, instead. Study on development of countermeasure techniques applicable for existing buildings is also underway.



	Liquefied sites	Non-liquefied sites
$F_L \leqq 1$	53 ●	35 △
$F_L > 1$	0 ●	24 △

Fig. 21 Comparison of liquefaction evaluation and observation results

4.6 Tsunami Evacuation Buildings

As for the design of buildings against tsunami force, the guidelines for tsunami evacuation buildings [11] are the unique technical information previously. These guidelines are established as part of the countermeasures for Tonankai-Nankai earthquake provided by the Central Disaster Management Council. In the guidelines, the tsunami force is considered to be

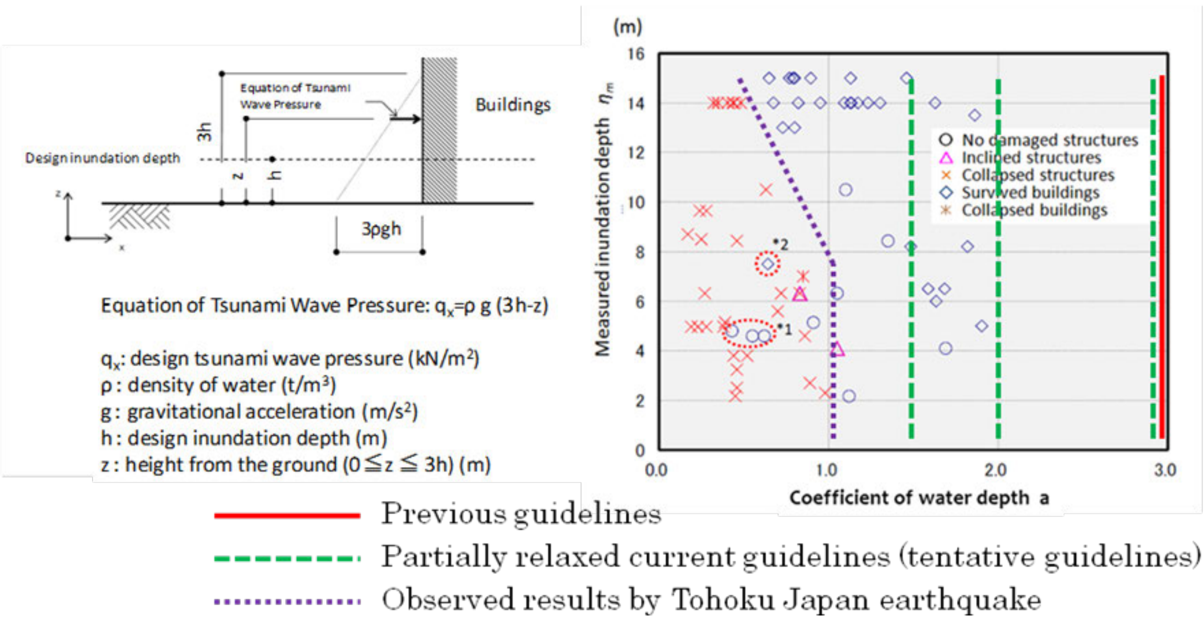


Fig. 22 Estimated coefficient of water depth of detailed studied buildings

equivalent static water pressure as shown in Fig. 22 (left) where the static water pressure of 3 times of the inundation depth is considered including the tsunami dynamic force. Here, 3 is the coefficient of water depth proposed by the waterway model test [12].

As explained above, about 100 buildings were carefully selected and studied in detail. First, the horizontal resistant strength of each building is evaluated whether damaged or not from the surveyed dimensions. Next, the coefficient of water depth is calculated so that the tsunami horizontal force estimated considering the observed inundation depth at or around the building as a function of the coefficient agrees with the calculated building strength. Fig. 22 (right) shows the relation of tsunami inundation depth and the estimated coefficient for the studied buildings. It can be seen that the coefficient of water depth is about 1.0 and it reduces as the inundation depth increases shown by dotted lines. In the tentative guidelines announced from the MLIT in December 2011, the coefficient of water depth was decreased by 2.0 in case the building was protected by the shading effect from front building and/or embankment. Further decreased value of 1.5 in case the building located at 500m or further from the coastline and river in addition to shading effect as shown by dashed lines is set.

The BRI worked with Kajima Co. and Univ. of Tokyo and conducted waterway experiment in 2012, and improved CFD (Computational Fluid Dynamics). In future, the improved CFD technique for evaluation of tsunami pressure on buildings will be used to increase the accuracy of the effect of openings of the buildings, the effect of water infiltration into the buildings and so on, which can further relax the design of tsunami evacuation buildings.

5. RESEARCH ACTIVITIES FOR KEEPING FUNCTION AND SEISMIC PERFORMANCE OF BUILDING

5.1 Higher level of performance for keeping function after earthquake

National and local government office buildings must be an emergency base and assist refugees and citizens just after earthquakes. But more than 10 local government buildings suffered from the 2011 Tohoku Japan Earthquake. There were damages of structural and non-structural member as shown in Fig. 23. They could not keep their function because staffs were prohibited to continuing to work inside. All of damaged buildings are designed by old building structural code, the judgment for safety or not depends greatly on the damage of non-structural members.

Fig. 23 Structural and non-structural damage of local government building

Another example is the damage of gymnasiums which must be utilized for places of refuge after earthquakes. A lot of gymnasiums suffered falling-down of ceiling (Fig. 5) and spalling of concrete at joins between steel roofs and RC columns (Fig. 4).

Because the damage on pile head of foundation as shown in Fig. 24, the super-structure tilted and the inhabitants could not continue to live in the building. The damage for buildings whose superstructure was retrofitted or strengthened was founded. These buildings will be demolished or jacked up after underpinning of piles.

Fig. 24 Structural damage of pile head

For performance for keeping functional after earthquake, it must be avoid these damages for especially facilities as the emergency base and assist refugees and citizens such as national and local government office, gymnasiums, schools, residences and so on. The required performance for these buildings will be made clear and technologies for an effective seismic retrofit will be developed.

5.2Detailed evaluation for earthquake resistance performance of buildings
On August 29, 2012, the estimation of earthquake damage by the mega-earthquake in Nankai trough was announced [9]. In the worst case, the number of completely collapsed buildings and broken buildings by fire will be about 0.95 to 2.38 million and the number of lost people will be 80 to 320 thousand.

Under the mega-earthquake, earthquake motion will be severe and the amplitude of motions will probably exceed the level of earthquake motion which is prescribed in the building structural code. Also earthquake motions observed during recent earthquakes sometimes exceeded the level in the code, as shown in Fig. 25. But severe damages in the areas were not always reported.

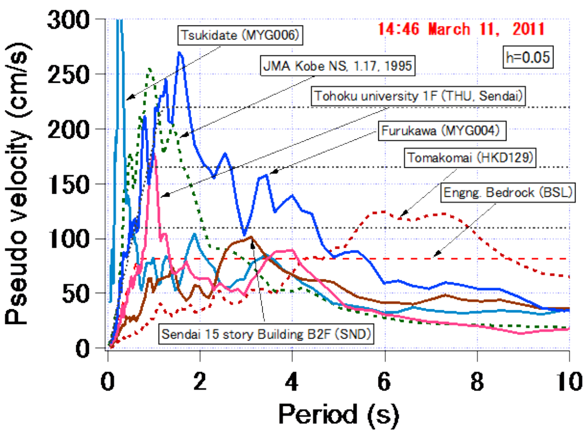


Fig.25 Pseudo velocity spectra of observed earthquake motions

In order to precisely evaluate the earthquake response and resistance of building, following items will be reviewed;
1) Dynamic soil structure interaction
Earthquake motions to building are changed by the kinematic interaction due to effects of embedment and pile foundation. Also the damping effects will be changed by radiation damping to ground through the foundation.

2) Structural modeling
More detailed modeling of structures is necessary to obtain the precise response of building. There are effects of presence of floor slab and restriction of column deformation on the response.

3) Material strength and proposed design formula
The design values of strength in every material and proposed design formula are set to a safety side. The values based on more actual characteristics must be estimated.

6. CONCLUSIONS

The BRI and the NILIM collaborated in the process of the recorded strong motions in and around instrumented buildings and also in the field survey of damaged buildings and residential land by the Tohoku Japan earthquake. The outlines of the on-going and near-future research activities for countermeasure against damages and so on are summarized.
1) Possibility of free access to the digital data recorded by the BRI strong motion network
2) Prediction of long-duration and long-period earthquake ground motion for design use and re-evaluation of structural performance under multiple cycles of loadings
3) Improvement of the seismic performance of fallen down of ceilings and so on
4) Liquefaction countermeasure for residential houses, for which neither structural calculation nor soil investigation is mandatory
5) Evaluation of tsunami force necessary for the design of tsunami evacuation buildings
6) Higher level of performance for keeping function after earthquake
7) Detailed evaluation for earthquake resistance performance of buildings

7. ACKNOWLEDGMENTS

The authors wish to express their gratitude to the members of the BRI and the NILIM who served for data analyses of observed strong motions and

field surveys of earthquake and tsunami damage of buildings and residential land, and to those of the building users/owners who allow site surveys under severe restoration conditions. Finally, the authors would like to express our deepest condolences to those who lost their families and those who are suffering from the disaster until this moment.

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Study of Seismic Force Coefficient for Rockfill Dams Based on Recent Seismic Motion Records

by

Takashi Sasaki[1], Hiroyuki Satoh[2] and Katsushi Aoi[3]

ABSTRACT

In 1991, a modified seismic coefficient method was proposed for the seismic performance evaluation of rockfill dams in Japan with a height less than 100m, in which the vertical distribution of seismic force is established with taking dam body's seismic response into account.

We studied the design rationalization of rockfill dams on the basis of the modified seismic coefficient method. This method can be used for a simple evaluation method for the seismic performance of rockfill dams as well. Drawing on many recent records of seismic motion occurring at dam sites, this paper examines the seismic force coefficient that represents seismic force in the modified seismic coefficient method and proposes a revised seismic force coefficient that can also be applied to rockfill dams with a height greater than 100m.

KEYWORDS: Earthquake, Modified seismic coefficient method, Rockfill dams, Seismic response.

1. INTRODUCTION

The seismic coefficient method, which is the current design standard used in rockfill dams in Japan, defines seismic force as a constant inertial force in the vertical direction [1]. This assumption, however, does not reflect actual rockfill dam behavior during earthquakes, thus making it difficult to achieve efficient design rationalization. The "*Draft of Guidelines for Seismic Design of Embankment Dams*" [2] (hereinafter referred to as the "*Draft of Guidelines*") was drawn up in June, 1991, as a seismic performance evaluation method for rockfill dams in preparation for a prospective design method with a more realistic seismic load and material strength. In the *Draft of Guidelines*, a modified seismic coefficient method is proposed as a seismic performance evaluation method for rockfill dams under 100m in height, in which the vertical distribution of seismic force is established with taking dam body's seismic response into account. In fact, the seismic force coefficient had been formulated prior to the implementation of the *Draft of Guidelines* through the examination of various data including eight events of relatively large seismic motion recorded at dam sites. But, since the implementation of the *Draft of Guidelines*, a number of seismic motion with large peak acceleration have been recorded at many dam sites. With the aim of realizing the design rationalization of rockfill dams using the modified seismic coefficient method, it became necessary to review the seismic force coefficient by referring to recent seismic motion records and examine the implementation to rockfill dams with a height greater than 100m.

2. OUTLINE OF THE STUDY

The seismic force coefficient in the *Draft of Guidelines* [2] was formulated through an examination of seismic motion recorded at dam sites in the 1980s and earlier in Japan. Following the implementation of the *Draft of Guidelines*, however, a number of large-scale earthquakes such as the South Hyogo prefecture Earthquake in 1995 have occurred and many seismic motion data with large peak accelerations have been recorded at some dam sites.

Furthermore, the seismic force coefficient in the *Draft of Guidelines* can be applied only to embankment dams with a height less than 100m.

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As for embankment dams with a height greater than 100m, the following explanation is noted in the *Draft of Guidelines*: embankment dams with a height greater than 100m tend to have a longer natural period that may be a significant factor in reducing the seismic force specified in the *Draft of Guidelines,* provided that the frequency characteristics of seismic motion in bedrock are taken into account [2].

In the light of these situations, the seismic force coefficient in the modified seismic coefficient method needs to be reviewed with reference to seismic motion records from dam sites in recent years. We gathered seismic motion records, including the latest data and analyzed these to select input seismic motions in order to examine the seismic force coefficient. The chosen seismic motions were used to investigate the seismic force coefficients for model rockfill dams with heights of 50m, 75m, 100m, 125m and 150m, respectively. Based on the results, we discuss the relationship between the dam height and the seismic force coefficient of rockfill dams including those with a height greater than 100m.

3. SELECTION OF INPUT EARTHQUAKE MOTIONS

Among 1,883 data of seismic motion recorded in bedrock or inspection galleries at dam sites from 1966 to 2008, those with a maximum horizontal acceleration exceeding 100 gal were selected. Thus, 48 seismic motions were selected as the input seismic motions. These are listed in Table 1. The histogram of maximum horizontal acceleration values of selected seismic motions is illustrated in Fig. 1, where most data are distributed in the range between 100 and 200 gal.

The relationship between maximum horizontal acceleration and maximum vertical acceleration for the selected 48 seismic motions is shown in Fig. 2. Although the ratios of maximum horizontal acceleration to maximum vertical acceleration are mostly plotted around 1:0.5, some data lie close to or beyond the 1:1 line. In view of the maximum vertical acceleration in Table 1, some relatively recent seismic motions recorded after 1997 are found to be in the ratio of approximately 1:1 and a tendency for the maximum horizontal acceleration to increase can also be observed after 1997.

The acceleration response spectra of horizontal seismic motions with a damping factor of h=5% and of vertical seismic motions are shown in Fig. 3 and in Fig. 4, respectively. The peak acceleration response spectra of the 48 seismic motions are in the range of between 0.1 and 0.3 seconds.

4. METHOD OF ANALYSIS

4.1 Outline

Equivalent linearization analyses [3] was conducted for the rockfill dam models using the complex response method to obtain the dam body's seismic response. We examined 20 circles on the upstream side [4] shown in Fig. 5 to calculate the seismic force coefficient (k/k_F) for each circle by dividing the average response acceleration by the maximum acceleration of input seismic motion. Here, k is the seismic force coefficient of a dam body and k_F is the design seismic intensity of the ground [2].

4.2 Analytical Models and Input Material Properties

The analytical models were rockfill dams with a central impervious core, and heights of 50m, 75m, 100m, 125m and 150m, respectively. The upstream and downstream slope gradients were determined with a stability analysis based on the seismic coefficient method [1] that is the present design standard in Japan, and the seismic coefficient was set at 0.15. The reservoir water level was set at 92% of the dam height and both the upstream and downstream gradients were calculated so that the minimum safety factor against sliding narrowly exceeded 1.2 [3]. The 100m-high dam model obtained is illustrated in Fig. 6. Other model dam shapes were decided in proportion to the 100m-high model dam. The specifications and reservoir water level of the model dams are listed in Table 2 and their finite element mesh is shown in Fig. 7.

Table 1. List of 48 Selected Seismic Motions

No.	Date	Name of Dam	Location of Seismometer	α_{xmax}(gal) ※1	α_{ymax}(gal) ※2	$\alpha_{ymax}\| / \|\alpha_{xmax}\|$	Name of Earthquake
No.1	1976 06.16	Miho	Observation Room of Water Leakage	−125.57	43.17	0.344	The Eastern Yamanashi prefecture Earthquake
No.2	1978 06.12	Tarumizu	Inspection Gallery Located at bottom Part	178.43	83.88	0.470	Earthquake Off Coast of Miyagi prefecture
No.4	1983 08.08	Miho	Observation Room of Water Leakage	−149.37	−54.60	0.366	Boundary in Mid-Kanto Earthquake
No.5	1986 06.27	Ishibuchi	Ground on the Right Bank	−180.30	※)No Records	−	The Southen Iwate prefecture Earthquake
No.6	1987 01.09	Tase	Inspection Gallery	103.40	30.97	0.300	The Northern Iwate prefecture Earthquake
No.7	1987.12.17	Nagara	Dam Foundation	−262.00	−86.00	0.328	Earthquake off the East Coast of Chiba prefecture
No.11	1987.12.17	Nagara	Ground on the Left Bank	−281.00	111.00	0.395	Earthquake off the East Coast of Chiba prefecture
No.13	1989.10.27	Sugesawa	Ground on the Right Bank	−101.36	−26.28	0.259	The Western Tottori prefecture Earthquake
No.14	1993 07.12	Pirika	Inspection Gallery	116.69	72.53	0.622	Earthquake off the Southwest Coast of Hokkaido
No.17	1994.12.28	Wada	Ground on the Right Bank	108.75	50.63	0 466	Earthquake far off the Coast of Sanriku
No.19	1995 01.17	Gongen	Foundation	103.67	−65.71	0.634	The South Hyogo prefecture Earthquake
No 20	1995 01.17	Hitokura	Lower Inspection Gallery	−182.13	62.86	0.345	The South Hyogo prefecture Earthquake
No 21	1995 01.17	Minoogawa	Inspection Gallery Located at bottom Part	−134.99	80.21	0.594	The South Hyogo prefecture Earthquake
No 22	1996 03.06	Miho	Observation Room of Water Leakage	−140.06	−73.63	0.526	The Eastern Yamanashi prefecture Earthquake
No 23	1997 03.16	Ameyama	Inspection Gallery	172.75	63.69	0.369	The Northeastern Aichi prefecture Earthquake
No 25	1997 03.26	Turuda	Inspection Gallery	−154.94	−71.44	0.461	The Northwestern Kagoshima prefecture Earthquakes
No 28	1997 04.03	Turuda	Inspection Gallery	−110.69	29.00	0.262	The Northwestern Kagoshima prefecture Earthquakes
No 31	1997 05.13	Turuda	Inspection Gallery	−109.00	62.13	0.570	The Northwestern Kagoshima prefecture Earthquakes
No 33	1997 08.23	Kasho	Inspection Gallery Located at bottom Part	117.61	117.46	0.999	The Western Tottori prefecture Earthquake
No 34	1997 09.02	Kasho	Inspection Gallery Located at bottom Part	−113.37	−48.18	0.425	The Western Tottori prefecture Earthquake
No 35	1997 09.04	Kasho	Inspection Gallery Located at bottom Part	344.02	−152.49	0.443	The Western Tottori prefecture Earthquake
No 36	1997 09.04	Kasho	Inspection Gallery Located at bottom Part	−244.24	−152.49	0.624	The Western Tottori prefecture Earthquake
No 37	2000.10.06	Kasho	Inspection Gallery Located at bottom Part	−528.49	485.21	0.918	The Western Tottori prefecture Earthquake
No 38	2000.10.06	Kasho	Inspection Gallery Located at bottom Part	−531.12	485.21	0.914	The Western Tottori prefecture Earthquake
No 39	2000.10.06	Sugesawa	Lower Inspection Gallery	−157.60	−108.74	0.690	The Western Tottori prefecture Earthquake
No 41	2000.10.06	Sugesawa	Ground on the Right Bank	−307.01	249.20	0.812	The Western Tottori prefecture Earthquake
No 42	2000.10.06	Takasegawa	Inspection Gallery Located at bottom Part	−106.20	70.93	0.668	The Western Tottori prefecture Earthquake
No 43	2000.10.07	Kasho	Inspection Gallery Located at bottom Part	133.82	−63.58	0.475	The Western Tottori prefecture Earthquake
No 44	2000.10.07	Kasho	Inspection Gallery Located at bottom Part	−113.25	−63.58	0.561	The Western Tottori prefecture Earthquake
No 46	2003 05.26	Tase	Dam Foundation	−232.09	117.72	0.507	Earthquake off the Coast of Miyagi Prefecture
No 47	2003 05.26	Hanayama	Ground on the Right Bank	237.20	−122.68	0.517	Earthquake off the Coast of Miyagi Prefecture
No 49	2004.10.23	Gejogawa	Inspection Gallery of the Central Lower	215.11	66.06	0.307	Mid Niigata Prefecture Earthquake
No 50	2004.10.23	Sabaishigawa	Lower Inspection Gallery	130.56	−81.35	0.623	Mid Niigata Prefecture Earthquake
No 51	2004.10.23	Shirokawa	Inspection Gallery Located at bottom Part	−161.55	−48.29	0.299	Mid Niigata Prefecture Earthquake
No 52	2004.10.23	Sabaishigawa	Lower Inspection Gallery	−231.20	224.39	0.971	Mid Niigata Prefecture Earthquake
No 53	2004.10.23	Shirokawa	Inspection Gallery Located at bottom Part	−191.73	78.80	0.411	Mid Niigata Prefecture Earthquake
No 54	2004.10.24	Shinyamamoto	Bedrocks in the Traverse Line B	609.15	182.47	0.300	Mid Niigata Prefecture Earthquake
No 55	2004.10.24	Shinyamamoto	Bedrocks in the Traverse Line B	−751.21	182.47	0.243	Mid Niigata Prefecture Earthquake
No 56	2004.10.27	Shinyamamoto	Bedrocks in the Traverse Line B	−371.82	−174.93	0.470	Mid Niigata Prefecture Earthquake
No 57	2004.10.27	Shinyamamoto	Bedrocks in the Traverse Line B	−682.55	−174.93	0.256	Mid Niigata Prefecture Earthquake
No 58	2005 08.16	Kejonuma	Dam Foundation	100.44	−39.31	0.391	Earthquake off the Coast of Miyagi Prefecture
No 59	2007 03.25	Hakkagawa	Foundation	166.78	166.78	1.000	Noto Hanto Earthquake
No 61	2007 07.16	Kakizakigawa	Foundation	−143.34	75.62	0.528	The Niigataken Chuetsu-oki Earthquake
No 62	2007 07.16	Sabaishigawa	Foundation	−129.46	84.44	0.652	The Niigataken Chuetsu-oki Earthquake
No 63	2007 07.16	Kochi	Foundation	291.50	−152.63	0.524	The Niigataken Chuetsu-oki Earthquake
No 64	2007 07.16	Tan-ne	Foundation	−157.25	86.88	0.552	The Niigataken Chuetsu-oki Earthquake
No 98	2008.6.14	Minase	Foundation	158.44	182.19	1.150	The Iwate-Miyagi Nairiku Earthquake
No 99	2008.6.14	Ishibuchi	Foundation(estimated)	−465.34	−621.39	1.335	The Iwate-Miyagi Nairiku Earthquake

※1 Maximum Horizontal Acceleration : Downstream Direction ＋, Upstream Direction −

※2 Maximum Vertical Acceleration : Upward Direction ＋, Downward Direction −

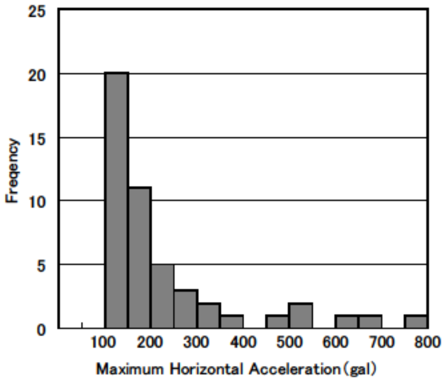


Fig.1 Distribution of Maximum Horizontal Acceleration

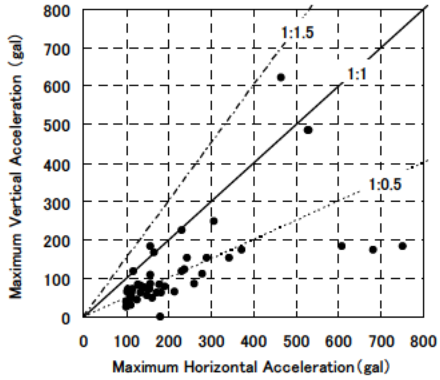
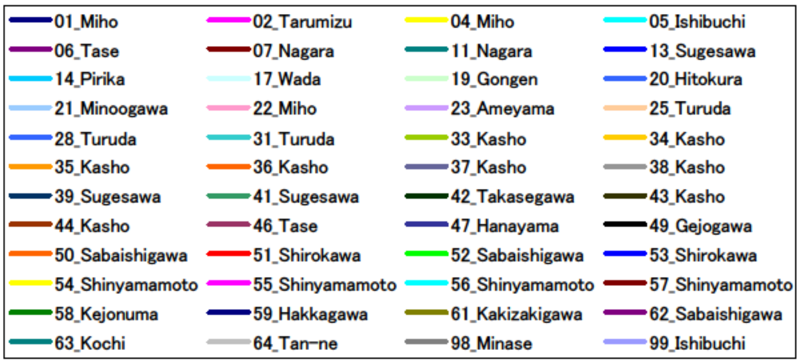


Fig.2 Ratio of Maximum Horizontal Acceleration to Maximum Vertical Acceleration



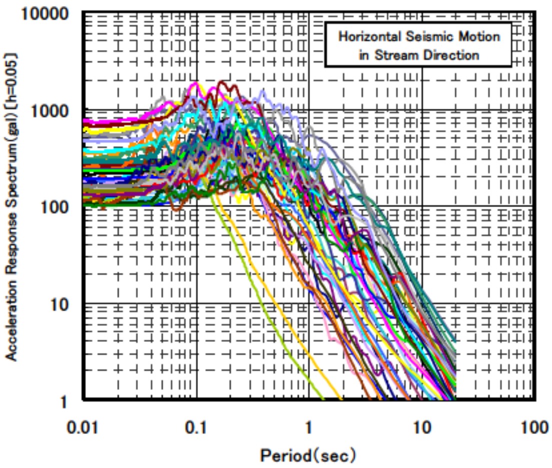


Fig.3 Acceleration Response Spectra in Horizontal Seismic Motion

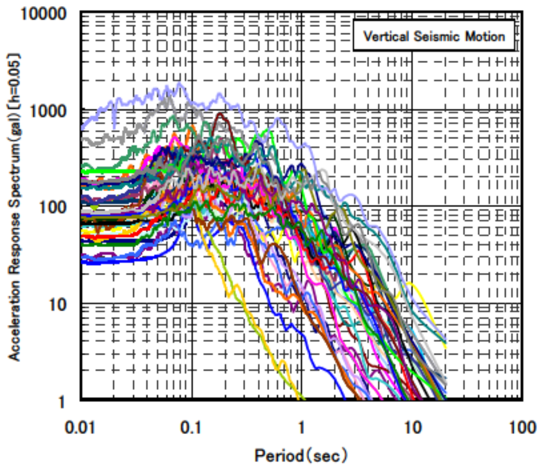


Fig.4 Acceleration Response Spectra in Vertical Seismic Motion

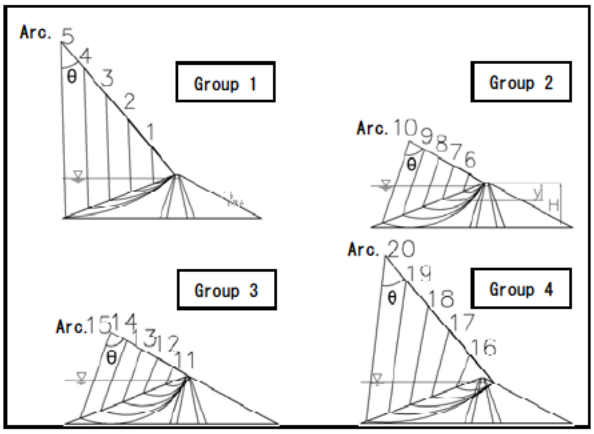


Fig.5 20 Sliding Circles for Analysis

Table 2. Analytical Models

Dam Height (m)	Crest Width			Slope Gradient		Zone Boundary Gradient		Reservoir Water Level (m)
	Total Width (m)	Core Width (m)	Filter Width (m)	Upstream	Downstream	Core	Filter	
50	5 0	3 0	1 0	1 : 2 6	1 : 1 9	1 : 0 2	1 : 0 35	46
75	7 5	4 5	1 5					69
100	10 0	6 0	2 0					92
125	12 5	7 5	2 5					115
150	15 0	9 0	3 0					138

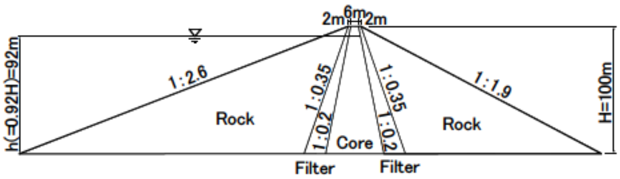


Fig.6 Analytical Model for 100m-High Dam Model

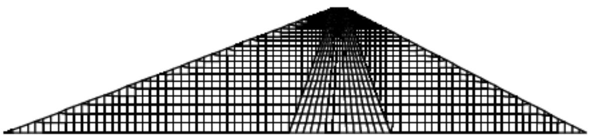


Fig.7 Finite Elements of Analytical Model

The input material properties used in the equivalent linearization method for seismic response analysis are summarized in Table 3 and Fig. 8. These material properties were set, based on the design values or test values of existing rockfill dam materials [3]. Energy dissipation from dam body to foundation was taken into consideration by adding an equivalent radiation damping ratio of 15% to the material damping ratio.

Table 3. The Input Material Properties used for the Equivalent Linearization Analysis

Material	Wet Density ρ_t(g/cm^3)	Saturated Density ρ_{sat}(g/cm^3)	Initial Shear Modulus G_0(MPa)※
Core	2.22	2.23	$\{60(2.17-e)^2/(1+e)\}\sigma_m^{0.7}$
Filter	2.13	2.24	
Rock	1.94	2.15	$\{93(2.17-e)^2/(1+e)\}\sigma_m^{0.6}$

※ e:Voio Ratio, σ_m:Mean Effective Principal Stress $\sigma_m=\{(1+2k)\nu\}/3$
k:Principal Stress Ratio (=0 5), ν:Poisson's Ratio(=0 35)

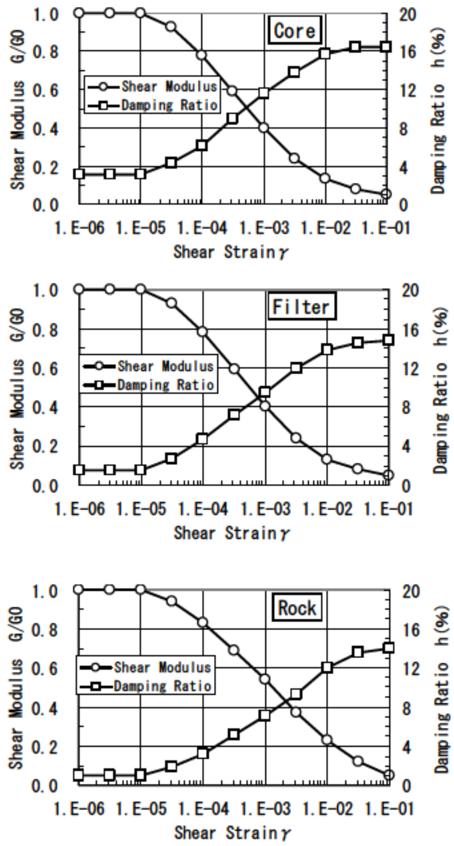


Fig.8 Dynamic Deformation Characteristics of Materials

5. RESULTS OF ANALYSIS

The results of the analysis of the model dams with heights of 50m, 75m, 100m, 125m and 150m are shown in Fig. 9. The height of circle (y) is defined as the vertical distance from the dam crest to the lowest point of a sliding circle. The height of circle (y) is nondimensionalized with the dam height (H). Fig. 9 indicates the relationship between the y/H value and the seismic force coefficient (k/k_F). We examined 20 sliding circles in Fig. 5, but no significant difference was detected in the four groups, and so the results from the analysis of Group 3, which mostly exhibited the largest seismic force coefficients in all groups, is taken as an example shown in this paper.

The results of the analysis of all dam height cases were compared with the seismic force coefficient in the *Draft of Guidelines*. It was observed that several seismic force coefficients at higher elevations exceeded that given in the *Draft of Guidelines*. This tendency is more clearly found in model dams with relatively low heights of 50m and 75m. With the exception of these cases, most of the seismic force coefficients were lower in value than that given in the *Draft of Guidelines*.

As shown in Fig. 10, the seismic force coefficients obtained in Fig. 9 were reorganized from the viewpoint of the statistical values of the mean (μ) and the standard deviation (σ). In the 50m-high model dam case, the value $\mu + \sigma$ of the seismic force coefficient at the crest (y/H = 0) was slightly larger than that given in the *Draft of Guidelines*. But in the other dam models, the values of $\mu + \sigma$ of the seismic force coefficients are smaller than those given in the *Draft of Guidelines* over the whole range of y/H. The values $\mu + 2\sigma$ of the seismic force coefficients are situated close to the envelope lines of maximum values and they exceed those given in the *Draft of Guidelines* in the high elevation area where y/H is smaller than approximately 0.4.

On the basis of these results, the relationship between height (H) and the values $\mu + \sigma$ of seismic force coefficient (k/k_F) according to y/H (= 0, 0.4 and 1.0) are illustrated in Fig. 11. To illustrate the k/kF distribution, we followed the *Draft of Guidelines*, in which values of y/H (= 0, 0.4 and 1.0) are drawn in a line graph. The values of $\mu + \sigma$ of k/k_F correlate well at the same y/H and the values of $\mu + \sigma$ of k/k_F decline linearly with the increase in the height of the dam models. Therefore, the seismic force coefficient can be calculated with a function of a dam height according to y/H and suggests the possibility of the seismic force coefficient being reduced by an increase in dam height. On the basis of the correction between dam heights and seismic force coefficients obtained in this paper, the approximations of the seismic force coefficients with dam heights as parameters were formulated. These are shown in Table 4.

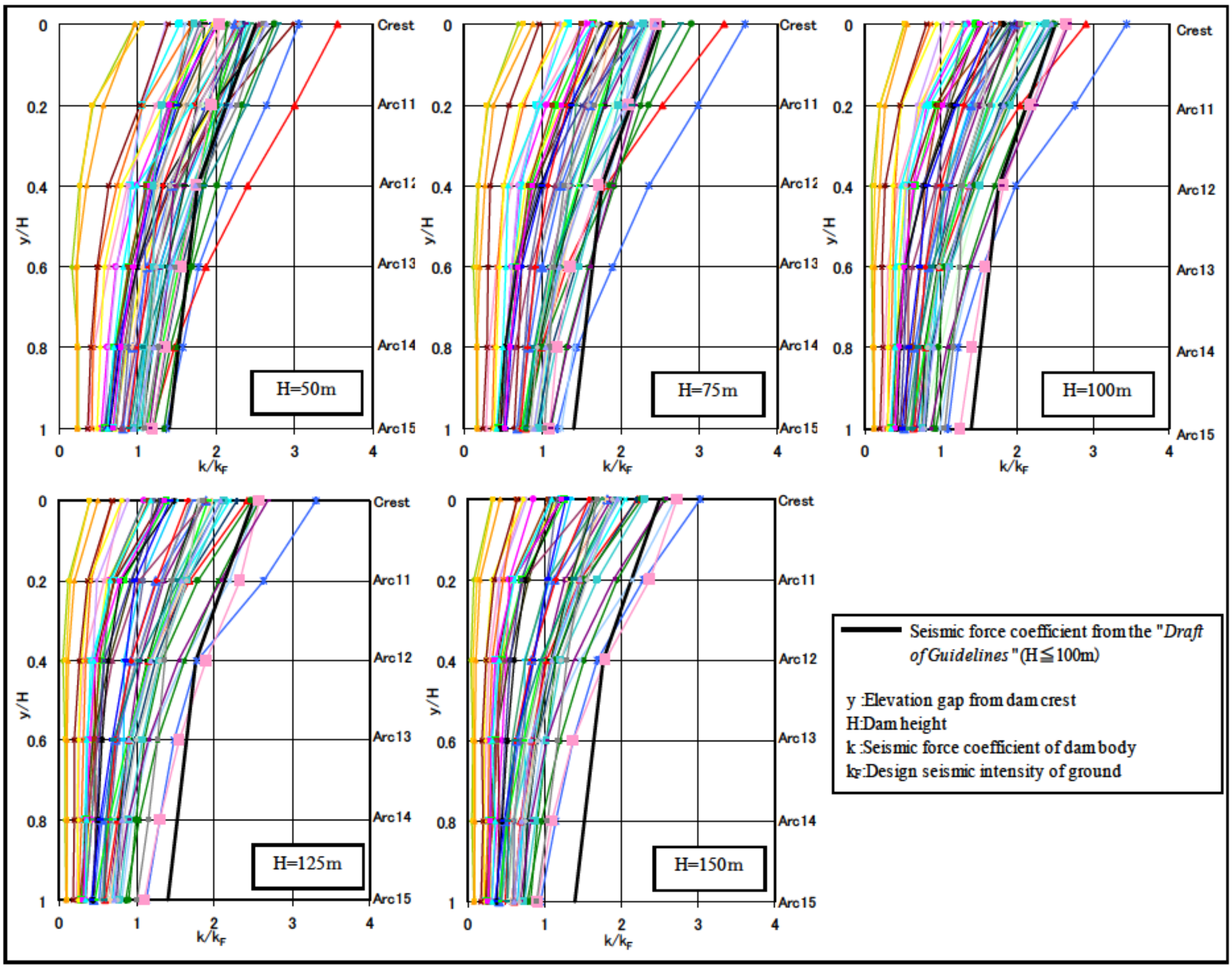


Fig.9 The Relationship between the y/H and the Seismic Force Coefficient (k/k_F)
(Results of Group 3)

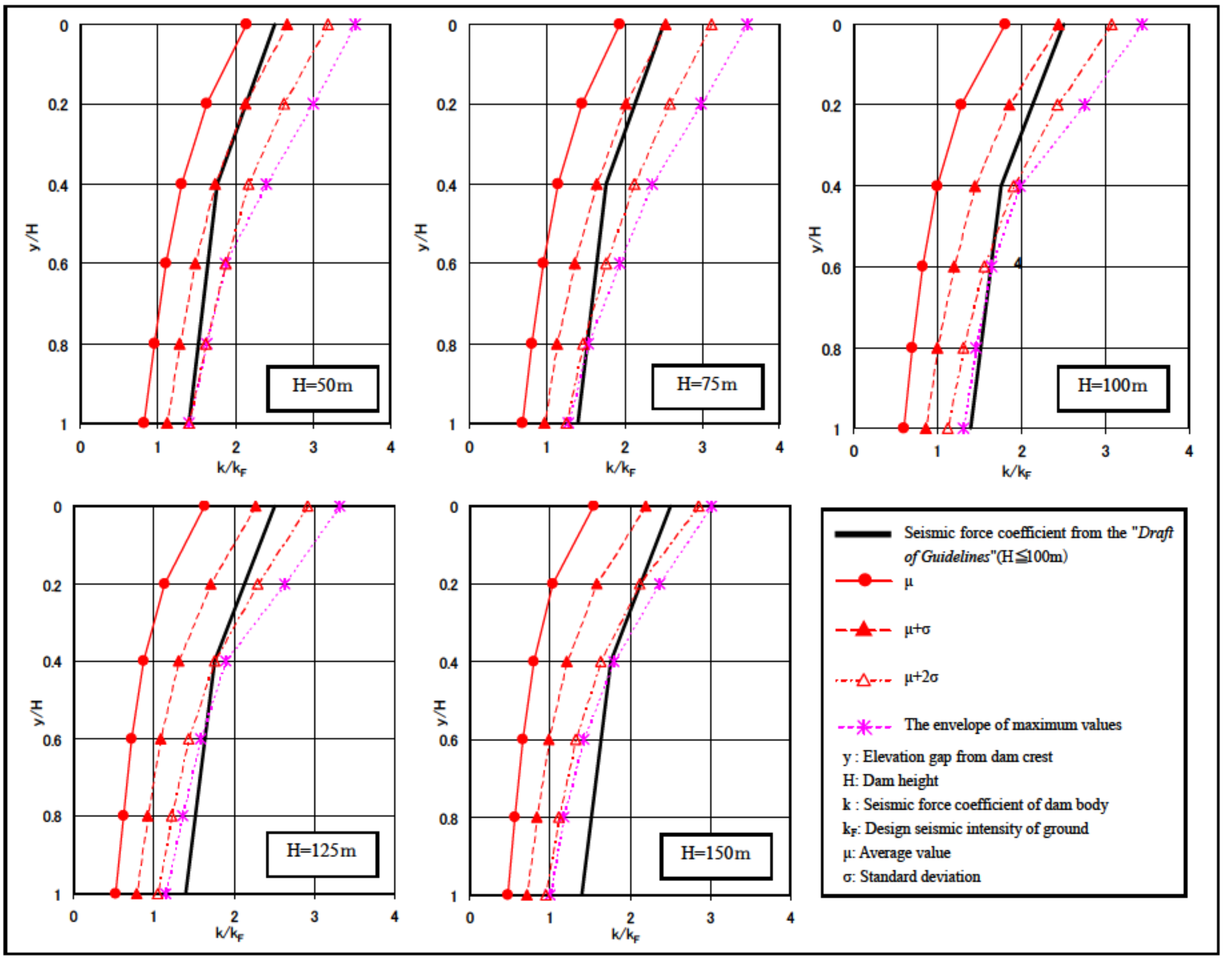


Fig.10 Statistical Analysis of Seismic Force Coefficient

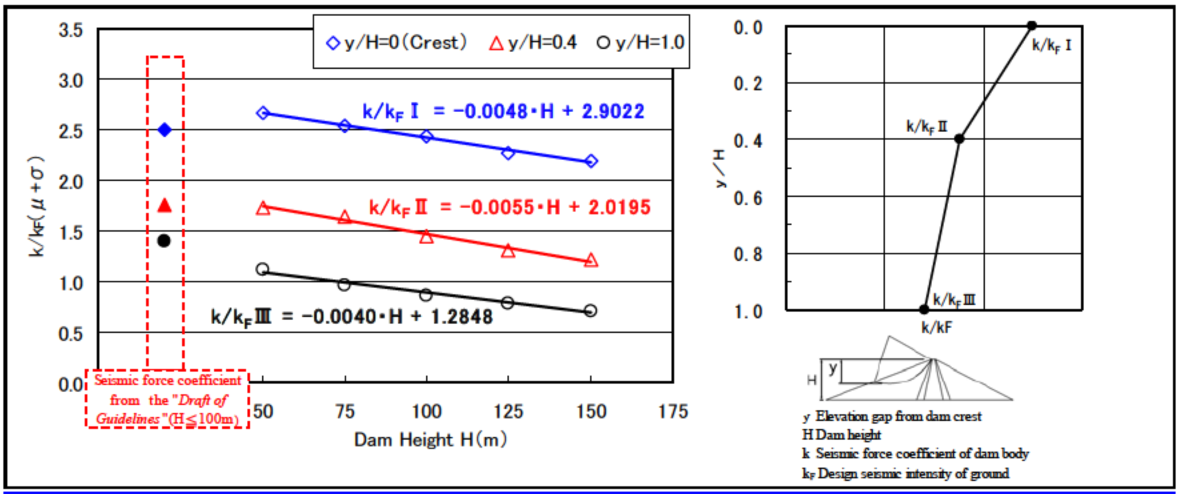


Fig.11 The Relationship between Height (H) and "μ + σ" of Seismic Force Coefficient (k/k_F)

Table 4. Approximation of Seismic Force Coefficient with Dam Height

y/H	Approximation of the seismic force coefficient
0.0（Crest）	k/k_F Ⅰ ＝-0.0048･H＋2.9022
0.4	k/k_F Ⅱ ＝-0.0055･H＋2.0195
1.0	k/k_F Ⅲ ＝-0.0040･H＋1.2848

k :Seismic force coefficient of dam body
k_F:Design seismic intensity of ground
k/k_F:Seismic force coefficient
H :Dam height（m）

6. CONCLUSIONS

In this paper, on the basis of seismic motion data recently recorded at dam sites in Japan, we examined the seismic force coefficient using the modified seismic coefficient method that has been promoted as a rational design method and a simple seicmic performance evaluation method for rockfill dams. As a result, we obtained the following findings.

(1) Recent seismic motion records were used to calculate seismic force coefficients for rockfill dam models with heights of 50m, 75m, 100m, 125m and 150m. The results of calculations were treated statistically and the values of $\mu + \sigma$ of the seismic force coefficients were found to be almost equal to or lower than that given in the *Draft of Guidelines*.
(2) High correlations appeared between the seismic force coefficients and the dam height in the range of dam height between 50m and 150m. It was also observed that the seismic force coefficient declines linearly with an increase in dam height. Based on these results, we formulated an approximation formula for the seismic force coefficient as a function of dam height. The proposed formula can be applied to those dams taller than 100m up to 150m described in this paper.

In order to establish and propose rational design methods for rockfill dams in accordance with the modified seismic coefficient method, the authors will make a further study on the seismic force coefficient by taking the design strength of rockfill dam materials into consideration [5].

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Report on Field Surveys and Subsequent Investigations of Building Damage Following the May 6, 2012 Tornado in Tsukuba City, Ibaraki Prefecture, Japan

by

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ABSTRACT

Field surveys were taken in the city of Tsukuba, Ibaraki Prefecture, Japan immediately after the tornado of 6 May 2012 in order to evaluate the state of damage to buildings and other structures. Photographs were also taken of damaged buildings, etc., immediately after the disaster. Beginning May 7, a detailed audit was made, focusing on the Hojo district of Tsukuba, and an evaluation was made of the damage to each structure based on the wind damage ranking. In addition, measurements were taken of the dimensions of structural members of damaged buildings, etc., and estimates were made of the wind velocity at which damage occurred based on the resistance of buildings, etc. Furthermore, there was an investigation into the causes of damage to wooden buildings, and an examination of the destruction mechanism of wind force based on floor plans, etc., of wooden houses that were collected after the 2011 Great East Japan Earthquake.

KEYWORDS: Tornado, Fujita Scale 3, Tsukuba

1. INTRODUCTION

At around 12:35 p.m. on May 6, 2012, a tornado formed in Joso City, Ibaraki Prefecture, Japan and moved toward the city of Tsukuba, where it caused extensive damage, particularly to buildings in the districts of Hojo and the Tsukuba North Industrial Park. An announcement from the Japan Meteorological Agency acknowledged that the phenomenon that caused this windblast was a tornado. The damage the tornado inflicted covered a 17 km-long area from Joso to Tsukuba, with a maximum width of 500 meters. Based on the degree of damage, it was estimated to be of the F3 class on the Fujita Scale. In addition, at approximately the same time, there were two other tornados, one that formed at Chikusei City in Ibaraki (ab. 12:30 p.m.), and one that formed in the city of Moka, Tochigi Prefecture (ab. 12:40 p.m.). The Chikusei tornado covered a distance of 21 km, had a maximum width of 600 m, and was estimated to be of the F1 class, while the Moka tornado covered a distance of 32 km, had a maximum width of 650 m, and was estimated to be of the F1 or F2 class.

2. SUMMARY OF DAMAGES

2.1 Damage Statistics

According to an announcement by Tsukuba city officials, one person was killed and 37 were injured, 209 buildings were completely destroyed, 47 buildings sustained major damage, 197 buildings sustained partial damage, and 639 buildings sustained light damage. There was also damage to public facilities (such as Hojo Elementary School, Tsukuba Kindergarten, Hojo Nursery School, municipal housing units, community centers, etc.), damage to the agricultural infrastructure (including facilities such as warehouses, "pipe houses," etc., crops, machinery, forest trees, scattering of debris on farmlands, falling hail), as well as power outages caused by broken utility poles (affecting about 21,000 households immediately after the disaster). There was also damage sustained by research labs, industrial facilities, etc., at the North Industrial Park.

2.2. Distribution of Damage

The National Institute for Land and Infrastructure Management of the Ministry of Land, Infrastructure, Transport and Tourism and the

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Building Research Institute started conducting surveys of affected sites immediately after the disaster to study the state of damage to buildings, etc.2.5) Starting on May 7 (the day after the disaster), each building, especially in the Hojo district, was surveyed, and evaluated based on the wind damage ranking system described earlier. In addition, damage to roofs that could not be seen in ground-based surveys was evaluated from interpretations of high-resolution aerial photographs taken by the Geospatial Information Authority of Japan on May 7. A total of 697 buildings were evaluated through the on-site surveys and aerial photo interpretations. A total of 548 of these buildings had damage of Rank 1 or greater, while the remaining 149 structures had no damage. Table 1 shows the number of buildings for each wind damage rank.

Figure 1 shows an enlarged map of the Hojo and Koizumi districts. Hojo, which has long been a densely built-up area, was the district sustaining the greatest amount of damage in Tsukuba.

Table 1 Number of buildings for each wind damage rank in Hojo distinct

Damage rank	Number of buildings
5	51
4	78
3	149
2	181
1	89
Total	548

Figure 1 Map of the distribution of damage in the Hojo district

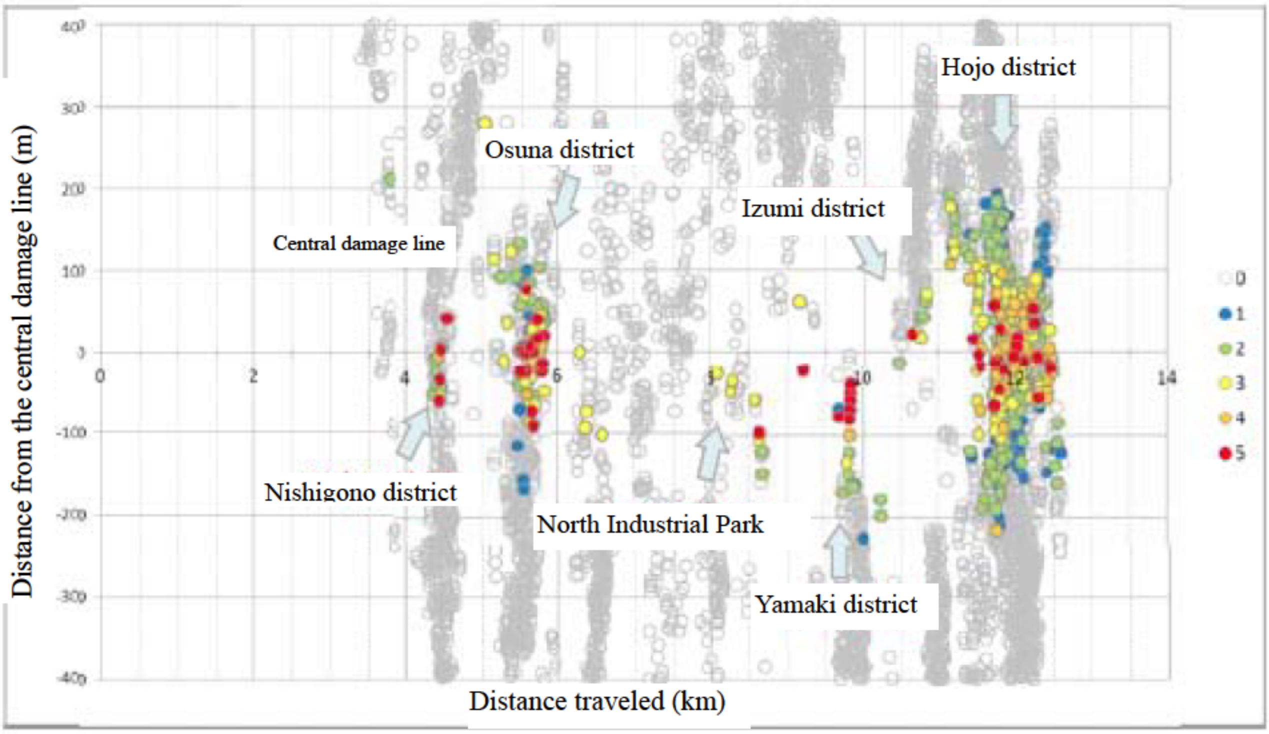


Figure 2 Distribution of building damage in Tsukuba city

Figure 3 Central damage line and central path of the tornado (Hojo district)

It appears that the distribution of damage in the Nishigono, Osuna and Hojo districts was concentrated along the central damage line, the damage in the North Industrial Park and the Yamaki and Izumi districts tended to be unevenly distributed on the underside of this line (the right side of tornado's path). In Figure 2.3-6, the buildings that were not damaged are designated with gray circles. In these latter districts, we can see that there are areas with vacant land near the exact central damage line, and areas with few buildings. This may account for the skewed damage distribution.

Figure 3 is an enlarged map of Hojo district. The green line in the figure is the central path of the tornado (funnel cloud) that was estimated from the tornado images taken by Dr. Miyagi from the area surrounded by dotted red line. The central damage line described earlier and the estimated central path of the tornado are roughly parallel to each other, with the central damage line proceeding about 35 m to the right of the central tornado path. As a result, it was estimated that the tornado rotated in a counterclockwise direction. Assuming the vortex of the tornado to be a Rankine vortex, and the central damage line to represent the greatest velocity of the tornado (the sum of the swirling flow vector and traveling speed vector), the maximum wind speed radius of this tornado in this area (inside the dotted red line in Figure 3) was estimated to be 35 m.

3. STATE OF DAMAGE TO BUILDINGS

3.1 Damage to Wooden Buildings

During the on-site survey conducted in Tsukuba City, Ibaraki Prefecture, information on tornado-induced damage to numerous buildings and other structures and objects was collected. This chapter presents an overview of damage patterns that were found in wooden, steel-frame and concrete buildings, as well as in non-buildings such as walls, automobiles, and trees.

(1) Overturned superstructure with foundation (one building)

A wooden superstructure in the Hojo district, was overturned with its foundation by the force of tornado. The condition of the second floor confirmed that it had been flipped upside down, but the roof truss and the first floor section could not be found (Photo 1). In addition, the mat foundation was overturned (Photos 2). There were almost no traces of the crushed stone beneath the foundation having been dragged (Photos 3). An aerial photo taken after the disaster confirmed that members on the right side of the tornado's traveling direction were scattered about (Photo 4).

Photo 1 Wooden structure overturned from its foundation

Photo 2 Relation between the building's position and the position of the overturned section

Photo 3 Condition of crushed stone under the foundation

Photo 4 Aerial photo of the same area after the disaster

Photo 5 Foundation (corner section) of the structure in Photo 7

Photo 6 View of the 1st story floor of the structure in Photo 7

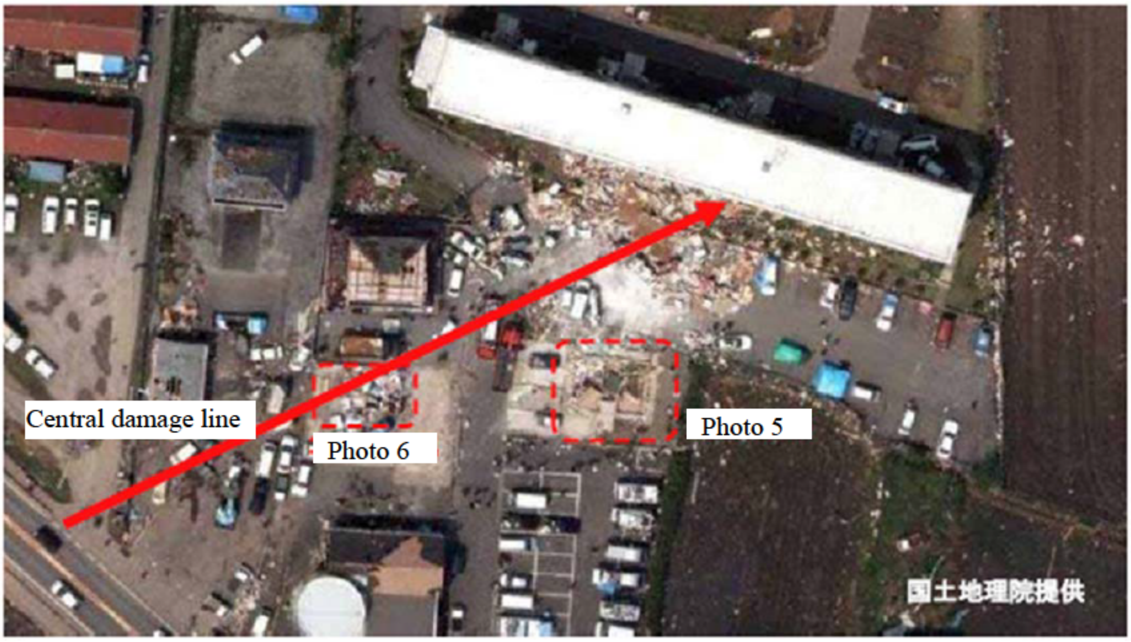


Photo 7 Aerial photograph of the same buildings after the disaster

(2) Scattering of the superstructure (6 buildings)
Several examples of the scattering of superstructures were found. Photos 5 and 6 show examples of scattering of superstructure, including the ground sill, while there were cases where only the floor framing of the 1st story, including the ground sill, remained. Aerial photos taken after the disaster (Photos 7) confirm that members of damaged buildings were crashed onto the reinforced concrete structure behind them.

(3) Collapse and other damage of superstructures
Numerous examples of buildings that had been collapsed were found. Some features of buildings (roofs, outer walls) remained, while nothing remained of the buildings. There were numerous examples of residual deformation caused. In Hojo district, the force of the tornado moved the superstructure of one building off of its foundation and into the adjoining street.

(4) Damage to roof trusses (about 70 buildings)
Numerous examples of damage to roof trusses were found. Wind pressure coefficients on the roofs are changed according to the shape of

Photo 8　A damaged gable roof

Photo 9　A damaged gable roof

Photo 10　A damaged hip roof

Photo 11　A damaged pent roof

Photo 12 Collapse of Steel-frame building

Photo 13 Residual deformation of Steel-frame structure

the roof. However, it was determined that damage was not related to roof shape, as it occurred in gable roofs of both relatively old houses (Photo 8) and relatively new houses (Photo 9), hip roofs (Photo 10), pent roofs (Photo 11), among others.

3.2 Damage to Steel-Frame Buildings

Information was collected on the pattern of damage to 3 steel-frame buildings that appeared to have structural damage. The present section focuses on damage to the structures themselves. Damage to exterior materials, roofing materials, etc., is examined in detail in Section 3.4.

Photo 12 shows a damaged 2-story steel-frame building. Its frame and foundation were moved, the frame was destroyed in the main-beam direction, and there was partial damage in the foundation. The superstructure of this building is believed

Photo 14 Displaced roof tiles

Photo 15 Metal roofing hung on power lines

Photo 16 Finishing materials torn off the underside of an elevated walkway

Photo 17 Damage to storefront glass

Photo 18 Damage to a 5-story apartment building

to have moved with the foundation after it was destroyed. The 1st story column capitals of the building were severely bent in the main-beam direction (weak axis), and column base bolts were severed in the base plate. The foundation was partially damaged, although a clear pattern could not be discerned.

Photo 13 shows a one-storied steel-frame structure that appears to have been a storage building. The deformation angle of about 1/10 (rad) in the ridge (longitudinal) direction of the steel frame was confirmed to have been residual deformation. There were only furring strips in the ridge direction of the structure, and no compressed beams were found. The shear bolts on the ends of the tensile braces in the ridge direction were severed. The location of the shearing, (top end or bottom end) differed among braces.

3.3 Damage to Reinforced Concrete Buildings

Within the range of the tornado in the present study, there were no frames of reinforced concrete structures that were found to have been damaged by the tornado winds. Also, as will be described in Section 3.4, in the area around a steel-reinforced concrete apartment building (Hojo housing built by Employment Promotion Corp.), there was major damage, such as scattering of superstructure members of the wooden buildings, but the only damage confirmed in the apartment building was at the door or window openings. Similarly to other steel-reinforced concrete buildings, no damage to the frames of this apartment building was found.

3.4 Damage to Building Exteriors

This section describes the main types of damage patterns to materials of building exteriors that were categorized as roofing materials, external wall materials, opening section members, etc. Exterior materials are the most vulnerable to tornado winds, and there were numerous examples of damage in the study area. In addition, at work facilities in the North Industrial Park, not only were the exterior materials damaged, but interior materials were also damaged at the same time, as well equipment systems installed outside of buildings.

The most common type of damage to roofing materials that was found was the displacement of roof tiles of wooden structures. This has also been one of the most common types of tornado damage seen in previous disasters. Photo 14 shows examples of such damage. Damaged roofing materials included not only tiles, but also such things as long sheet metal roofing, which was found hanging from power lines, etc., or which had fallen or crashed onto nearby houses (Photo 15).

In steel-frame office facilities in the North Industrial Park, damage to exterior siding, framework ceilings, etc., was found (Photo16). In addition, finishing materials on the underside of an elevated walkway at a medical facility were also damaged.

Photo 17 shows damage to glass shop windows. Most of this damage was caused by either wind pressure of tornado or by impacts from flying debris. The arrow in Photo 17 shows damage that is believed to have been caused by flying debris.

Photo 18 shows damage to opening and other areas of the Hojo housing built by Employment Promotion Corp. (completed in October 1984). This is a 5-story reinforced concrete housing unit. As can be seen in Photo 7, there was remarkable damage extending to all floors facing the south of the building. Looking at the damage to lower stories, we found that much flying debris had accumulated on 2nd story verandas, and cracks were found in the reinforced concrete rail in the central on 1st floor in the ridge direction. As a result, it appears that damage was caused by numerous pieces of flying debris. In contrast, there was less accumulated debris on the 4th and 5th floor verandas, but the rail near the central on 4th floor in the ridge direction was yanked outward. From this condition, there is a possibility that near the central part in the ridge direction, especially on the upper floors, there was an extremely large negative

Photo 19 Damage to interior materials

Photo 20 Damage to a ceiling

Photo 21 Remarkable deformation to the roof of a bicycle parking area

Photo 22 Broken and tilting utility poles

Photo 23 Overturned passenger cars

Photo 24 Toppled tree

pressure that developed as the tornado approached. However, the aluminum sashes on the north side showed the same type of damage, and numerous aluminum sashes of small, movable windows were also knocked out. Furthermore, it can be confirmed that objects such as bedclothes inside these apartments were moved toward the outside.

3.5 Damage to Interior Materials

Office facilities in the North Industrial Park sustained damage to exterior materials, which was found to have led to damage to interior materials like ceiling materials

(Photos 19 and 20). Some office furniture such as desks and chairs were overturned. However, no damage was found in the structure skeletons of these buildings.

3.6 Other Damage

Besides buildings in the study area, there were numerous examples of damage to other types of structures, automobiles, and trees, among other things.

Photos 21 shows damage to a bicycle parking area whose roof shows wave-like destruction. Bending of road signs can be seen, while Photo 22 shows a series of utility poles that were toppled.

In the study area, numerous automobiles were overturned (Photo 23). In addition, not only light passenger cars but also relatively heavy vehicles like sedans and trucks were overturned. A tree in Photo 24 was toppled and the skin of the tree was peeled off.

4. ESTIMATION OF WIND VELOCITY BASED ON DAMAGE PATTERNS IN BUILDINGS

In the following investigation for buildings and other structures that were confirmed to have been damaged by the tornado, several calculation assumptions such as the power generated by the horizontal rotation flow and a sudden drop in atmospheric pressure, the mechanism for destruction of buildings, the weight of buildings, longitudinal sections of structural members, etc., were used as a basis for estimating the wind velocity that attained the mechanism for destruction of buildings, and the wind velocity at which the windward end of foundations and floor sections of structures began to rise.

4.1 Evaluation of Wind Force

Generally, the wind forces that act upon buildings during a tornado are considered to be the following:

i) Forces generated by horizontal rotational flow
ii) Forces generated by a sudden drop in atmospheric pressure near the center of tornado
iii) Impact forces from flying debris (not examined in the present study)

In i), a uniform horizontal flow is assumed to act upon a building, and the force of the wind pressure ω (N/m^2) is expressed with the following equation. Here, ρ is the air density (1.2 kg/m^3), V (m/s) is the wind velocity of the tornado, and C_f is the wind force coefficient. V is added to the calculation as a vector representation of the wind velocity of the horizontal rotational flow and the velocity at which the tornado progresses.

$$\omega = 0.5\,\rho\,V^2 \times C_f$$

On the other hand, in the case of ii), which includes things that are not considered in the wind force calculations which assume a uniform horizontal flow, for cases where the center of a tornado passes near a building, vertically uplifting suction forces resulting in the sudden drop in air pressure caused by the tornado act upon the entire building, including all parts of the roof, etc.

It should be noted that in the present estimations, several considerations were made for the buildings that were examined for the overturning moment resulting from wind force: integral values of wind pressure from the average height of the roof to the ground surface, and of wind pressure at the range of roof surface for the vertical direction (excluding the range of the eaves). Each of these wind forces is considered to act upon a building. For the horizontal direction of buildings that were studied for horizontal force resulting from wind force, the integral value of the wind pressure from the building height to the center of the 1st story was considered to act as wind for upon a building in the same way as normal wind force calculations. In addition, for structures that were examined for the overturning moment resulting from wind force, the integral value of wind pressure from the structure height to the ground surface was considered to act as wind force, in the same way as on buildings, while the wind force acting in the vertical direction was ignored.

4.2 Collapsed Wooden Buildings

The building in Photo 1 was a 2-story wooden structure located in the Hojo district of Tsukuba City. The 1st story section was destroyed. Nearby were such structures as a tilting steel-frame building (Photo 13) and wooden buildings whose superstructures were blown away while their foundations remained (Photo 6 and 7).

From measurements taken of overturned foundations (estimated from laser distance measurements and photographs), it was estimated that the floor area was 6.5 m span× 9.2 m ridge. The roof was a tiled hip roof with an estimated pitch of 16.7 degrees. The length of the eaves was estimated to be 0.455 m (one side) based on comparisons between roof surface and floor area from an aerial photograph and based on standard dimensions of wooden buildings. From this information, the average roof height from the ground surface was calculated to be 6.45m. Regarding the destruction mechanism, it was assumed that the overturning center was the base of foundation on the downwind side on the structure plane of span (the base of foundation in Photo 3), and that a height H of 6.65m was the summation of the average roof height from ground, 6.45 m and 0.2m, the thickness of the floorboards of the foundation. In addition, the weight of the building was calculated to be 630.5 kN.

(1) Cases where only horizontal rotational flow of a tornado acts upon a building

For the wind force calculated using the wind force coefficients (horizontal and vertical directions) in a uniform horizontal flow for the building, when the wind force reached the overturning resistance moment, the wind velocity is calculated as follows:

$$M_w = W \times (D/2) \times 1000 \quad \text{(Nm)} \tag{1}$$

$$M_h = B\int_{H'}^{H} (0.5\rho C_{fh} V^2)x\,dx \quad \text{(Nm)} \tag{2}$$

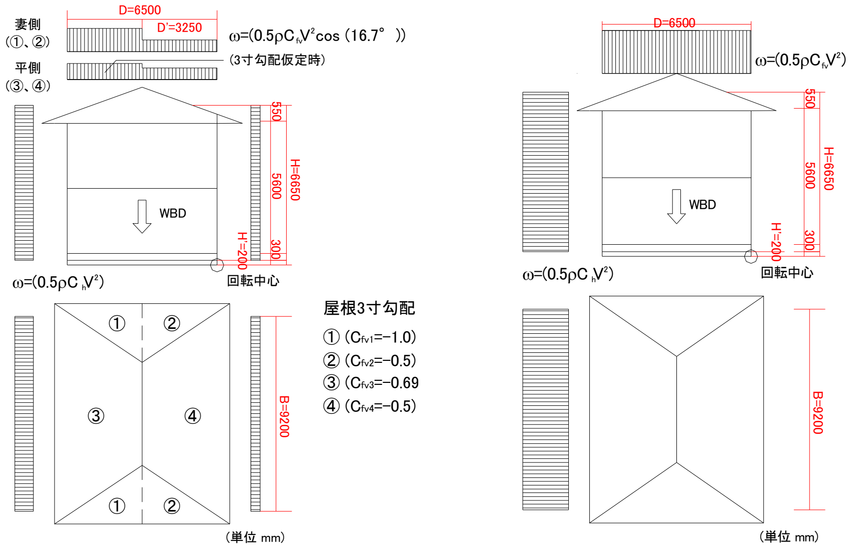
$$q = 0.5\rho V^2 \quad \text{(N/m}^2\text{)} \tag{3}$$

$$\begin{aligned} M_v &= \int_0^{D/2} 2C_{fv2} q\cos(16.7^\circ)x^2 dx + \int_{D/2}^{D} 2C_{fv1} q\cos(16.7^\circ)x(D-x)dx \\ &+ \int_0^{D/2} C_{fv4} q\cos(16.7^\circ)x(B-2x)dx + \int_{D/2}^{D} C_{fv3} q\cos(16.7^\circ)x(B-2D+2x)dx \\ &= \left\{\frac{1}{12}D^3C_{fv2} + \frac{1}{6}D^3C_{fv1} + \left(\frac{1}{8}BD^2 - \frac{1}{12}D^3\right)C_{fv4} + \left(\frac{3}{8}BD^2 - \frac{1}{6}D^3\right)C_{fv3}\right\} q\cos(16.7^\circ) \quad \text{(Nm)} \end{aligned} \tag{4}$$

The calculations were made using ridge length B (m), span length D (m), and height H (m). Also, total building weight is W (kN), and thickness of the floorboards of the foundation is H', with overturning resistance moment M_w (Nm), overturning moment due to horizontal wind force M_h (Nm), and overturning moment due to vertical wind force M_v.

Here, the overturning resistance moment M_w around the basic rotation center of the building was calculated by multiplying total building weight W by half of the span span length D of the building (Eq.1). On the other hand, the overturning moments due to wind force (M_h and M_v) were calculated by integrating the roof area of the building (B×D) with the wind pressure (0.5 ρ × V^2×C_f, or C_{fv1}, C_{fv2}, C_{fv3}, C_{fv4}) upon the wall surface in the building's ridge direction (B ×(H-H')) (Eqs. 2, 3 and 4). Here, q is the velocity pressure (N/m2). C_{fh} is the wind force coefficient of the wall surface of the building's ridge direction, and C_{fv1}, C_{fv2}, C_{fv3}, C_{fv4} are all coefficients of vertically uplifting wind force acting upon hipped roofs.

Figure 4(a) shows the distribution of wind pressure. When the vertical wind force coefficients were set as C_{fv1}, C_{fv2}, C_{fv3},and C_{fv4}, the hypothetical external pressure coefficients acting upon buildings with hip roofs were assumed as C_{fv1} = -1.0, C_{fv2} = -0.5, C_{fv3} = -0.69, and C_{fv4} = -0.5. Furthermore, when the



(a) Case of action by only horizontal rotating force (b) Case of action by sudden drop in air pressure

Figure4 Hypothetical distribution of wind pressure

$$M_w = W \times (D/2) \times 1000 \qquad \text{(Nm)} \qquad (5.3.5)$$

$$M_h = B\int_{H'}^{H} (0.5\rho C_{fh} V^2) x\, dx \qquad \text{(Nm)} \qquad (5.3.6)$$

$$M_v = B\int_{0}^{D} (0.5\rho C_{fv} V^2) x\, dx \qquad \text{(Nm)} \qquad (5.3.7)$$

hypothetical wind force coefficient C_{fh} of the wall surface of the building's ridge direction was 1.2, the wind velocity at the start of overturning is estimated

$V = 97$ (m/s) (F4)

(2) Cases where a sudden drop in atmospheric pressure acts upon a building, in addition to (1)

In addition to the wind force generated by the horizontal rotational flow, when a passing tornado causes a sudden drop in air pressure, the wind velocity at the time when the cumulative effect of these forces results in the overturning resistance moment of the building being reached is calculated as follows.

In this case, values of the vertical wind force coefficient C_{fv} and the wind force coefficient C_{fh} of the wall surface of the building's ridge direction were obtained from wind pressure tests that utilized the equipment for generating tornado-like air flow conditions. However, the experimental results were obtained under test conditions that were limited to those listed in Reference 1. Generally, the value of the wind force coefficient is considered to depend on various conditions such as the ratio between the radius of the rotational flow core and the building dimensions and the traveling velocity of the tornado. Therefore, as a wind force coefficient of the wall surface of the building's ridge direction, the investigation was conducted using the coefficient 1.2 which is used when a uniform lateral flow is assumed (this was 60% of the wind force coefficient C_{fh} of the results of pressure experiments that utilized the tornado-like conditions generator). In these investigations, the pressure acting upon the floor surface of the mat foundation was assumed to be roughly equal to inner pressure of the model obtained from the pressure experiments, and the pressure difference (between that acting on the roof surface and that on the foundation floor) when the tornado was passing through was considered to be the maximum vertical uplifting value.

Similar to the investigation in (1), the overturning resistance moment Mw around the rotation center of the base of the building was calculated by multiplying the weight of the building W by half of the of the span length D (Eq.5). On the other hand, the overturning moments due to wind force (M_h and M_v) were calculated by integrating the wind pressure ($0.5\rho V^2 \times C_{fh}$ or Cf_v) which acting upon the roof surface ($B \times D$) of the building and the wall surface of the building's ridge direction ($B \times (H - H')$) with respect to areas (Eqs. 6 and 7).

Figure 4(b) shows the hypothetical wind pressure distribution that was used here. Based on the results of the experiments for generating tornado-like rotational flow in Reference 2 (see Reference Figure 1 and Reference Table 1), the wind force coefficient for vertical uplifting from C_{pe} (x/R ≒ 1.0) was assumed to be -1.8. When the tornado-like rotational flow acted on the building anticlockwise, and when wind force coefficient C_{fh} of the wall surface of the building's ridge direction was assumed to be 2.0 or 1.2, each wind velocity at the start of overturning was estimated respectively,

<u>V = 68 m/s</u> (for wind force coefficient C_{fh} of 2.0 on the wall surface of the building's ridge direction),

or

<u>V =77 m/s</u> (for wind force coefficient C_{fh} of 1.2 on the wall surface of the building's ridge direction).

(Reference 1)
Assumptions for calculating building weight

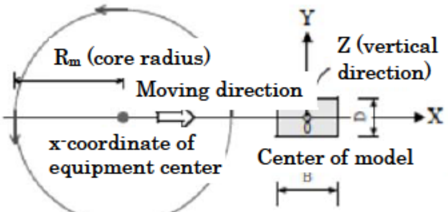
＊ Wall: Outer wall: siding, inner wall: plasterboard (estimated from flying debris)
＊ Roof: Hip roof, tiles (estimated from flying debris), 3-sun pitch, eaves of 0.455 m
＊ Foundation: Mat foundation (considered to be used in ordinary wooden houses, with assumed elevated height of 0.3 m, width of 0.15 m, floorboard thickness of 0.2 m)
＊ Loaded weight: 0.3 kN/m^2 (half of normal loaded weight for calculating earthquake force)
＊ Interior wall line: 3 in the ridge direction, 4 in the span direction (estimated from the build-up in the foundation)
＊ Exterior wall opening ratio: 1st floor 30%, 2nd floor 20% (assuming an ordinary wooden house)
＊ Average height of the roof from the building's foundation: 6.45 m (= 0.3 m + 2.8 m × 2 + 0.5 × 1.1 m) (see Fig. 4)
＊ Total building weight: 630.5 kN

(Reference 2)

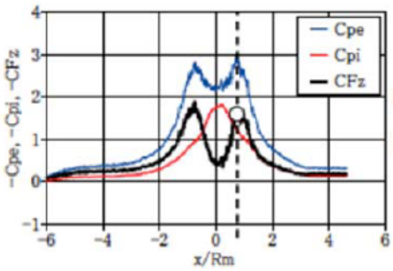
＊ Definition of coordinates, etc., in Reference, experimental conditions and results of wind pressure experiments
＊ C_{pe}: External pressure coefficient of vertical uplifting, C_{fy}: wind pressure coefficient of the wall (Y direction)
＊ (C_{pr}: Internal pressure coefficient; C_{Fz}: wind power coefficient of vertical uplifting; C_{Fx}: wind pressure coefficient of the wall surface (X direction)
＊ x: central coordinate of the tornado-like air flow generator moved for the center of the model;
＊ Rm: core radius of rotational flow

Reference Table 1. Experimental conditions (experimental scale of 1/250)

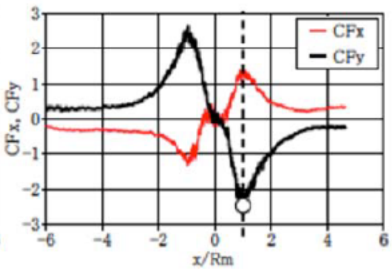
Model dimensions	Span D	98 mm
	Ridge B	152 mm
	Eaves height	49 mm
	Roof pitch	1/12 (gabled roof)
Core radius of rotational flow R_m		0.36 m
Ratio between model dimensions and core radius of rotational flow $(BD)^{0.5}/R_m$		0.34
Maximum tangent velocity V_m of rotational flow		9.6 m/s
Traveling velocity of the equipment		0.15 m/s



Ref. Fig.1 Definition of coordinates, etc.



(a) External pressure coefficients of vertical uplifting



(b) Wind force coefficients of wall surfaces

5. CONCLUSIONS

The National Institute of Land and Infrastructure Management (NILIM) and Building Research Institute (BRI) conducted investigations into the damage to buildings and others caused by the tornado occurred on May 6, 2012 in Tsukuba City, Ibaraki Prefecture. Estimation of wind speed at the time of damage occurrence and damage causing mechanism as well as patterns and distribution of the damage through on-site studies were also examined. The results can be summarized as follows:

(1) Statistics and distribution of damage
Damage of buildings and others in Tsukuba City was evaluated according to the high wind damage scale and the results were summarized in a damage distribution map, and then the distribution patterns were studied. The radius of maximum wind speed of the tornado which was supposed according to Rankine vortex models was estimated at about 35 meters based on damage distribution in Hojo area, Tsukuba City.

(2) Patterns of damage to buildings and others and damage causing mechanism
As for the major patterns of damage to buildings induced by the tornado, following data were obtained from on-site studies; those to wooden houses were overturning, scattering and collapse of upper structure, collapse of roof systems, breakage and peeling of exterior materials (roof materials, exterior walls and opening materials). Those to steel-framed and reinforced concrete buildings were breakage, peeling of exterior materials (roof materials, exterior walls and opening materials), etc. Damage was also extended to interior materials and outdoor equipment. As damage examples other than buildings, data of damaged structures, automobiles, trees, etc. were gathered.

The mechanism to cause damage to wooden buildings was examined based on assumed damage patterns. With regard to two-story wooden housing, wind speed causing collapse (failure of ultimate horizontal strength) and overturning are compared in their destruction modes.

(3) Estimation of wind speed based on patters of damage to buildings, etc.
Wind speed at the time of damage was estimated by assuming collapse mechanism of damaged buildings and structures, wind speed coefficient, etc. The relation between the distance from the tornado center and the estimated velocity was then studied. The results showed good correspondence with the wind speed distribution of tornado based on Rankine vortex models.

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ESTABLISHING DESIGN CRITERIA FOR ALL EXTREME LOADS (MULTI-HAZARD) FOR TRANSPORTATION INFRASTRUCTURE

by
W. Phillip Yen[1] and George C. Lee[2]

ABSTRACT

Under a Federal Highway Administration (FHWA) research contract at the Multidisciplinary Center for Earthquake engineering Research (MCEER), reliability-based bridge design principles and approaches for establishing Multi-Hazard Load and Resistance Factors Design (MH-LRFD) are explored. A theoretical framework to systematically establish important load combinations is developed (20). The objective of this short paper is to outline this framework and to briefly describe the major challenges of the on-going research project without mathematical formulations and results. Several relevant publications to this project including a few currently under preparation by the researchers are given in the Bibliography.

1.0 INTRODUCTION

The currently used AASHTO LRFD specifications are a reliability-based approach with the design limit states calibrated only for dead load and frequent live load. When the frequently applied loads are combined with infrequent extreme hazard loads, the probability-based methodology used to establish the AASHTO LRFD cannot be readily used. In professional practice today bridges are typically proportioned by using the LRFD and checked for strength against extreme load effect(s). The latter are available in different forms including several guide specifications published by AASHTO. Relative importance among regular loads and extreme loads and their various combinations is not known unless all loads are considered on the same platform.

Since 2008, with the support of FHWA, a research program has been carried out at MCEER which explore guiding principles, analysis and design approaches to consider all frequent and infrequent load effects on the same reliability-based platform, so that failure probabilities of the bridge due to individual loads and their combinations may be compared, and design limit states may be further developed for those cases the risks are not negligible. A theoretical framework is established to target the establishment of Multi-Hazard (MH) LRFD that are compatible with the current LRFD. In this formulation, a number of significant challenges have been identified that must be overcome, and certain assumptions and simplifications must be made and quantitatively justified.

Due to the lack of statistical data of extreme

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hazard loads (which are most likely time variables and the corresponding bridge damage/failure information), there exists a fundamental question whether or not MH-LRFD is necessary and can be successfully accomplished today). Furthermore, certain extreme hazards do not use force as the basis of design (e.g. scour is capacity-based and earthquake is moving towards performance-based.) Yet structural reliability is force-based consideration. Recognizing these facts, the objectives of using the MH-LRFD platform may be regarded as (1) to have a common ground to compare and evaluate all the possible individual and combined load effects on a bridge (or bridge components) so that those load effects with relatively low risk may be ignored in bridge design established on a quantitative base; (2) to pursue those important load effect combinations and to systematically improve the AASHTO LRFD extreme event design limit states; and (3) to identify and recommend important research opportunities for future study. This paper briefly summarizes the objectives and challenges of this current MCEER research project.

2. BRIDGE RELIABILITY

2.1 Bridge reliability under frequent loads

The current AASHTO LRFD is based on the consideration of bridge reliability, which theoretically should also be suitable for most MH loads. In general, the basic relationship between bridge failure probability p_f and reliability p_r is

$$p_r \quad 1- p_f \tag{1}$$

which implies that to consider the reliability is equivalent to consider the failure probability

The basic formula of bridge failure probability is

$$P(L \geq R) \quad p_f \tag{2}$$

where L is maximum load effect and R is resistance, both are random variables (RV). The case ($L \geq R$) is an event. That is, equation (2) implies the probability of such event is the failure probability. From (2), the load and resistance factors can be systematically determined. The established procedure to obtain the load factors from (2) is briefly summarized in the following:

Suppose L and R follow normal distributions, a standardized variable β can be specified directly relating to the failure probability p_f. That is, with known p_f , β is uniquely determined. It is defined as the reliability index.

β is a function of the means and standard deviations of L and R. Therefore, with known β, as well as the variation of L and R, the exact relationship between mean values of L and R, denoted as μ_L and μ_R, respectively are given as

$$\mu_L \quad \eta\, \mu_R \tag{3}$$

where η is a proportional coefficient.

The mean values of L and R are proportional to the design nominal values, denoted by NL and NR, and the proportional coefficients are known. Generally, we have

$$B_{(.)}\, N_{(.)} \quad \mu_{(.)} \tag{4}$$

where $B_{(.)}$ $N_{(.)}$ and $\mu_{(.)}$ are bias, nominal values and mean value of load (.)

From the relationship between mean value of L and R described in (3) and (4), the relationship between the nominal design values can be written as

$$\gamma \, NL \quad \Phi \, NR \qquad (5)$$

where NL and NR are respectively the nominal load and resistance.

Practically speaking, the load can be a combination of dead and live load, whose design nominal values are denoted as DL and LL. Usually, the ratio between DL and LL are also known. We can uniquely rewrite (5) as

$$\gamma_D \, DL + \gamma_L \, LL \quad \Phi \, NR \qquad (6)$$

Equation (6) is referred to as the design limit state equation, and γ_D , γ_L and Φ are the load and resistance factors. They directly and uniquely represent the bridge reliability. These factors quantitatively and qualitatively express the physical implications of the safety factors, used in ASD. This is an attractive feature because the bridge designers will have more confidence.

2.2 Bridge reliability under frequent and extreme loads

If the loads L are not random variables but sequences of random variables (random process), there are several challenges that need to be addressed before establishing the load and resistance factors. We do not have sufficient information on the intensity and frequency of occurrence of extreme loads and the corresponding damage/failure models of bridges.

To address bridge reliability among various frequent and infrequent loads that are random processes, we need to reconsider the formulation of bridge failure probability. In equation (2), L is the maximum value of load, which can be a single type of load; it can also be a load combination. A major difficulty is how to calculate the load combination with some loads that are time variables.

Although the dead load is time invariant, live load is time variable. The reason that dead load and live load can be added directly in the formulation of the AASHTO LRFD is because there is only one time variable load. In the case of more than one time variable loads, unless all the data of the possible time histories and amplitudes of all those loads are available, the reliability index cannot be directly obtained. Because of the lack of data, what we can do is to provide a “best” estimate to establish the reliability indices.

The best estimation can be made through a process called partial failure probabilities. This method separates these loads under certain conditions. After the separation, we will have several sub-cases and in each sub-case we only have one time variable load. In so doing, each sub-case is exactly like the situation of dead plus live load and this process can lead to a partial failure probability. The total failure probability is the sum of these partials failure probabilities.

$$p_f \quad p_{f1} + p_{f2} + p_{f3} + ... \qquad (7)$$

In equation (7), the second subscript 1, 2, 3, … stands for the first, second, third, … type of loads, which can be the dead load plus one single type of load, or they can also be the dead load plus combined loads, where the combined loads mean pure load combinations without the chance of one type of load being single.

In so doing, each partial failure probability can be used to determine a partial reliability index β_i and equations similar to (3) in format can be obtained

$$\mu_{L1} \quad \eta_1\, \mu_R \quad \mu_{L2} \quad \eta_2\, \mu_R \quad \mu_{L3} \quad \eta_3\, \mu_R \qquad (8)$$

Here, the subscripts L_1, L_2 and L_3, etc. are load effects, for example, L_1 can be DL + LL, L_2 can be DL + EQ, L_3 can be DL + LL + EQ, etc.

From equation (8), with a few additional simple steps, we can obtain the required design limit state equations dead, live and earthquake loads as:

$$\gamma_D\, DL + \gamma_L\, LL + \gamma_E\, EQ \quad \Phi\, NR \qquad (9)$$

Since these load effects are calculated together, equation (9) is therefore a reliability-based design limit state equation, in which all loads are considered equally in their probabilistic contributions to the failure of a bridge. The concept of all-inclusive effect will provide comprehensive bridge reliability, which is comparatively more rigorous and the resulted load factors should be more accurate.

3. SELECTION LOADS

The second challenge to establish MH-LRFD is to determine the loads that should be considered for bridge failure and those that may be neglected.

One of the feasible criteria for load rejection is the value of partial reliability. Generally, if a partial failure probability is κ times smaller than the allowable failure probability, the corresponding load or load combination may be rejected. This criterion may be expressed as:

$$p_{fi} \leq \kappa\, p_f \qquad (10)$$

where $\kappa \quad 0.1$ is considered to be a reasonable value by the researchers after certain simulations (not given herewith).

The advantage of using (10) is to significantly simplify the set of limit equations without scarifying the design accuracy.

4. EQUIVALENT LOAD EFFECT

The third challenge to formulate MH-LRFD is for important hazards that directly affect the bridge capacity such as the foundation movements, fire damage and bridge scour. To include scour in formulating the bridge failure probability as an example, it is necessary to transform its capacity effect to equivalent load effect. In the following, scour effect is briefly addressed.

With the presence of scour, the resistance of the bridge, R will be reduced, say, by ΔR. Therefore, equation (2) is re-written as

$$P(\, L \geq R - \Delta R\,) \quad p_f \qquad (11)$$

in which ΔR is also a random variable.

Equation (11) can be further rewritten as

$$P(\, L + \Delta R \geq R) \quad p_f \qquad (12)$$

where the reduction of bridge resistance can be treated as an equivalent load ΔR, based on which we can determine the corresponding "load factor" $\gamma_{\Delta R}$

Furthermore, we can find the relations between the reduction ΔR and the scour depth D_C , which is usually a design parameter when bridge scour is considered. It can be shown that the mean values of ΔR and D_C, denoted as $\mu_{\Delta R}$ and μ_C , have a deterministic relation given by

$$\mu_{\Delta R} \quad f(\mu_C) \tag{13}$$

With the help of (13), we can have an equivalent load factor γ_{CD} for the nominal scour depth CD. Therefore, the bridge scour hazard may be included into the total bridge reliability design.

The above concept can be extended to other non-force based effects. In so doing, all the significant natural hazards can be included in a uniform formula, the formula of bridge reliability.

5. RANGE OF ACCEPTABLE RELIABILITY

The load and resistance factors are established through certain bridge component reliability. They should not change for different bridge designs. However, variations in design will always exist (different bridge types and/or dimensions). Therefore, design sensitivity analysis should be conducted by varying the size, the material type, the span, the height, and other bridge parameters, denoted by BP, to see how the load and resistance factors change.

$$\delta(\, BP) \rightarrow \delta(\, \gamma_i, \Phi) \tag{14}$$

If the load and resistance factors (γ_i, Φ) are fixed, then the reliability will vary, that is

$$\delta(\, BP) \rightarrow \delta(\, \beta) \quad \Delta\beta \tag{15}$$

The challenge is the need for a criterion to quantify the result of sensitivity study. The researchers are using the variation range of reliability indices. With a variation of the bridge design parameters and with fixed value of load and resistance factors, the reliability index will change. Suppose β is the desired reliability index, with the variation, we will have β_U and β_L (denoting the upper and lower limit of the indices). Therefore, the difference, or the range of reliability index, is given by

$$\Delta\beta \quad \beta_U - \beta_L \leq [\Delta\beta] \tag{16}$$

This range must be limited to within a certain level, denoted by [(.)].

There is a need to simplify the complexities involved in formulating the design limit state equations. With different loads and their combinations, and types of bridge components, the resulting design limit state equations will yield large numbers of different values of reliability indices. Conversely, with a fixed value of reliability index, the number of corresponding limit state equations can be significantly large, which is not convenient, nor necessary for practical applications. The challenge is with acceptable range of reliability indices, we must try to reduce the sets of limit state equations for practical bridge design applications.

Based on the above approach, the reliability index will be limited to a reasonable range so that the design limit state equation can be suitable for the design of specific bridge components. Furthermore, this approach will simplify the design limit state equations.

6. LOAD IMPORTANCE FACTOR

Different weighting functions or importance factors have been used to take care of the relative importance of specific situations and/or consequences in establishing the demand for bridge design. For example, importance factors are used for different types and/or locations of bridges. Another example is the weighting function of different type of seismic regions for earthquake resistant design of structures.

These weighting functions have been used primarily from the viewpoint of the relative importance of the bridge capacity to damage/failure. From the viewpoint of MH loads, due to the significantly large differences of their amplitude and occurrence rate, large differences among the load factors after the failure probability analyses will occur. Because all the loads are considered on the same platform, these differences in load factors in the limit state equations will not alter the designs too much. However, when the load condition and/or the types of bridge component changed, these loads must be reconsidered. This will result in many extra limit state equations. By considering the weighting functions of loads, the sets of limit state equations will be reduced.

There are several reasons for considering different weighting functions for extreme loads. First, the failure of a bridge or a bridge component has not been rigorously defined. The consequence of a special "failure" of different location and of different type can be rather distinct.

Secondly, the cause of a bridge failure due to different loads can receive rather different public opinions. For example, the public may be more tolerance of a bridge failure due to certain extremely rare natural hazard loads, but be more critical of the failure due to regular loads.

To emphasize the importance of load specification, the researchers recommend the concept of load importance factors. As an example, denoting the load importance factors for dead, truck and earthquake load effect by $I_{(.)}$. I_D, I_T and I_E, respectively, the load importance factor I(.) on both sides of equation (4) for these three loads will not change the relationship between the nominal and mean values of a load, namely

$$I_{(.)} B_{(.)} N_{(.)} \quad I_{(.)} \mu_{(.)} \qquad (17)$$

This multiplication will affect the final determination of the load and resistance factors. To establish the values of the load importance factor is a challenging process, but it is essential in establishing design guidelines.

It should be noted that, while the load importance factors affect the load factors, they virtually do not appear in the design procedure. Instead, they are used for the purpose of code-generation.

7. SUMMARY

The AASHTO LRFD is based on the realization of bridge reliability. It specifies the values of loads, as well as designs the resistance of bridge according to acceptable failure probability. When a bridge only subjects to dead and live load, the failure probability is calculated and the bridge reliability analysis is carried out fully with reasonable accuracy. For engineering practice, the corresponding load and resistance factors are all calibrated.

Bridges at various locations will be subjected to other extreme loads for which the bridge reliability becomes far more difficult to model. An incremental approach has been used to artificially include those loads with partial safety factors, based on engineering experiences and judgment. In other words, in so doing, the factors of dead and live load are obtained through reliability analysis and others are obtained by using different approaches. This mixed method is a departure from the track of rigorous bridge reliability analysis. In certain cases, such designed bridge is not sufficiently safe while in other cases, the design is not cost-effective.

There is a need to handle MH-loads on the same platform with the regular loads. That is, all hazard loads applying on a bridge, as long as they can affect the bridge safety, should be equally considered. All the loads factors should be calculated based on the entire bridge failure probability. To do this, several significant challenges are facing the researchers. This short paper briefly summarizes these difficulties and the approaches that are being pursued to address these challenges by the researchers.

ACKNOLEDGEMENTS

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Outline of Japanese Design Specifications for Highway Bridges in 2012

by

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ABSTRACT

This paper presents the outline of revisions of the Japanese design specifications for highway bridges issued by Ministry of Land, Infrastructure, Transportation and Tourism (MLIT) in February 2012, and the commentary for the specifications was published by Japan Road Association (called “JRA” in the following text) in March 2012 [1]. The revised specifications incorporated the latest research achievements, many lessons learned from the recent earthquakes including the 2011 Great East Japan Earthquake and the durability related damages of existing bridges. Based on these lessons, design earthquake ground motions corresponding to the c -type earthquake were revised, and the requirements for easy and secure inspection and repair works for the bridges were clearly specified.

KEYWORDS: Japanese Design Specifications for Highway Bridges, Maintenance, Seismic Design

1. INTRODUCTION

Japanese Design Specifications for Highway Bridges (JRA specifications) are applied for Japanese road bridges and consist of five parts: Part I Common, Part II Steel Bridges, Part III Concrete Bridges, Part IV Substructures, and Part V Seismic Design. These specifications have been revised several times on technical progress and changes of social needs. In recent years, the 1996 specifications were revised to enhance seismic design mainly triggered by severe damages suffered from the 1995 Hyogo-ken Nanbu, Japan, earthquake. In the 2002 specifications, the performance-based design concept was introduced, and design requirements were clearly specified and the conventional detailed design methods including analytical methods and the allowable limits were used as verification methods and the examples of acceptable solutions. Additionally, the design considerations for durability were improved so as to design the sustainable structures [2].

The 2012 revised specifications were issued by (MLIT) in February 16, 2012, and the specifications and the commentary for the specifications was published by JRA in March. These revised specifications are improved based on the technical research achievements in terms of safety, serviceability and durability of bridges. These examples include introducing of integral abutment bridges and the use of higher strength rebar in comparison with conventional one. Moreover, many lessons learned from the recent earthquakes such as the 2011 Great East Japan Earthquake and from the damages of existing bridges due to aged deterioration have also been accumulated. Based on these lessons, design earthquake ground motions corresponding to the interplate-type earthquake were revised, and the requirements for easy and secure maintenance (inspection and repair works) for the bridges were clearly specified. This paper summarizes the main points of revisions in the 2012 specifications.

2. FUNDAMENTAL PREICIPAL OF MAINTENANCE

There are about 650 thousand road bridges which

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bridge length are 2m or more in Japan and aging rapidly. Focusing on the road bridges which bridge length are 15m or more (approx. 160, thousand bridges), there are approximately 30% of the bridges more than 40 years after construction, and approximately 50% of the bridges more than 30 years after construction as shown in Figure 1. As aged bridges increase, the bridges with damage due to deterioration such as fatigue, salt damage, alkali-silica reaction (ASR) have been increasing. However, budgets for maintenance, which is needed to keep road bridges healthy for a long time, keep decreasing so that it is important to reduce the maintenance and operation costs through countermeasures such as preventive maintenance of highway bridges.

The performance of the road bridges has changed by applying for various factors such as live load, seismic, and environmental effects in their service period. Therefore, it is important to perceive change of bridge condition by inspection and to be repaired or retrofitted as it needed timely and surely. However, it is not easy to do these things because most of existing bridges are difficult to inspect due to design concept without a view to inspecting, lack of inspection equipments such as inspection ladders, walkways, workspace, and so on. Consequently, a lot of existing bridges where appropriate measures have not been made remain even if damage of the bridge such as the corrosion of end of girder and bearing become significant.

On the basis of these lessons, it is clearly required as a fundamental principal of bridge design that structural systems of which maintenance is expected to be difficult and insecure should be avoided. It is also required that the bridges should be designed in consideration of maintenance methods such as periodic or emergency inspection, and repair, retrofitted works. The maintenance equipments such as inspection ladders, walkways, as shown in Figure 2, shall be provided to access easily and securely as they need. Visual inspections near the important structural parts are effective to be judged the bridge safety and quickly not only at normal inspection to use for a long time but also at emergency event such as the extreme earthquake. These equipments are helpful for the visual inspection more easily and securely. Strengthening of main girder in advance for temporary jack up is effective to replace the bearing in the future. Consideration of temporary support for replacement of bearings in the structural design is also effective.

There are a lot of existing bridges with unknown structural details such as foundation type, bar arrangement, especially in old bridges. In these cases, it is very difficult not only to examine the performance of the bridge appropriately but also to examine effective measures to repair or retrofit. Therefore, it is clearly required that various records on bridges about the investigation, design, construction, quality control should be preserved accurately and succeeded following stages to utilize not only for construction but also for maintenance. These kinds of information are indispensable to examine performance evaluation, repair or reinforcement method of the bridges in-service period.

3. FUNDAMENTAL PRINCIPAL OF DESIGN

In recent years, severe damages such as fracture of diagonal bridge bracing in steel truss bridges as shown in Figure 3, severe fatigue cracking of steel main girders, and fracture caused by corrosion of prestressed concrete bridge tension members were occurred in Japan. Fortunately, no bridge collapse has occurred while the I- 35W bridge fell in Minnesota, U.S. in 2007. The collapse of the I-35 bridge implies that fracture of specific member might cause the catastrophic damage of the bridge. Therefore, it is enhanced that the bridge should be designed not to collapse of whole bridge caused by damage of such critical members. For example, from a point of view of redundancy, in design of abutment foundation where located on the slope, it is recommended that the number of piles is arranged in more than four piles and more than two rows. This is because the abutment supported by multi rows of piles is more stable even if slope might be collapse due to landslide.

4. SEISMIC DESIGN

4.1 Revision of Design Earthquake Ground Motion Corresponding to Interplate-Type Earthquake

The Japanese design specifications for highway bridges consider two levels of earthquake ground motion (Level 1 and Level 2) and two types in Level 2 earthquake motion (Type I and Type II). Level 1 earthquake motion represents ground motion highly probable to occur during service period of bridges and its target seismic performance is set to have no structural damage. Level 2 earthquake motion is defined as ground motion with high intensity with less probability to occur during the service period of bridges. The target seismic performances against Level 2 earthquake motion is set to limited damage for function recovery in short period for high importance bridges and to prevent fatal damage for bridges such as unseating of a superstructure or collapse of a bridge column for standard importance bridges. Type I of Level 2 earthquake motion represents ground motion from large-scale interplate-type earthquakes, while Type II from near-field shallow earthquakes that directly strike the bridges.

In the revision in 2012, the Type I of Level 2 earthquake motion was revised considering earthquake motions from the 2011 Great East Japan Earthquake as well as the anticipated great earthquake along the Nankai Trough, of which the occurrence impends [3].

Design earthquake motions for highway bridges are set by multiplying zone factor, which will be described later, to the standard acceleration response spectra. A damping factor of 5% is considered. The standard acceleration response spectra are set for each soil profile type as shown in Figure 4. The soil profile type I, II, and III correspond to stiff, medium, and soft soil conditions, respectively. Type I earthquake motion is based on the ground motion in Tokyo area during the 1923 Kanto Earthquake (M_w =7.9). They had been introduced into seismic design of highway bridges in 1990, prior to Type II in 1996, and were revised for the first time in 2012 using recently developed attenuation relationships, and the strong motion records during the 2011 off Tohoku, Japan, earthquake (great east Japan earthquake, M_w =9.0) as well as the 2003 off Tokachi, Hokkaido, Japan, earthquake (M_w=8.0). Response spectra specified in the previous specifications are larger in soft soil (Soil profile type III) and smaller in stiff soil (Soil profile type I) because damage of structures by large earthquakes prior to the Kobe earthquake tends to be more significant in soft soil condition, while the relationship is reversed because earthquake motions recorded during recent large earthquakes show the intensive ground shaking tends to be more amplified in stiff soil condition than in soft soil condition.

Zone factors for Type I earthquake motion are also revised along with the standard acceleration response spectra. There had been three zones, A, B, and C, with zone factors 1.0, 0.85, and 0.7, respectively, and they had been employed for both Level 1 and 2 earthquake motions. As shown in Figure 5, zone A was divided into two zones, A1 and A2, as well as zone B into B1 and B2, while zone C was not changed in this revision. Zone factor for Type I earthquake motion, c_{Iz}, was set to be 1.2 for zones A1 and B1, 1.0 for A2 and B2, and 0.8 for C.

Figure 6 presents source regions of major plate boundary earthquakes that are taken into account in the revision. The moment magnitude M_w of off the Pacific coast of Hokkaido and Tokai-Tonankai-Nankai-Hyuganada earthquakes are assumed to be 9.0 besides off the Tohoku earthquake. Zones A1 and B1 were set based on the area where ground motion intensity is estimated larger than that in Tokyo area during the 1923 Kanto Earthquake.

Figure 7 compares acceleration time history, which is used for dynamic response analysis for seismic design, of Type I earthquake ground motion before and after the revision. Very long duration is considered based on the record obtained from the 2011 Great East Japan Earthquake.

4.2 Design Considerations of Effect of Tsunami, Large-scale Landslide, etc. on Structural Planning of Bridges

In recent earthquakes occurred in Japan, extreme events associated with a large earthquake, but not strong earthquake shaking, have caused collapse of bridges as shown in Figure 8. A bridge was collapsed by large-scale landslide around its abutment during the 2008 Iwate-Miyagi inland, Japan, earthquake [4], and many bridges were washed away by extreme tsunami during the 2011 Great East Japan Earthquake [5]. Although the large fault movement did not cause fatal damage to bridges in Japan recently, that caused fatal damage to bridges in the 1999 Chi-Chi, Taiwan, earthquake and the 1999 Kocaeli, Turkey, earthquake.

Although the extreme events listed above have critical effect on the performance of bridges, these events are not directly considered, but the effect of a strong earthquake motion is only considered in the seismic design of bridges according to the Japanese design specifications for highway bridges. This is because design philosophy for these events, which means the scale of external force considered, and the required performance, etc., has not yet been determined. Therefore, only design considerations to mitigate the effects of these events have been introduced in this revision.

Against tsunami, in particular, it is specified in the specifications that the local plan for disaster prevention shall be considered in planning of road, and in structural planning and structural design of bridges. For prevention of collapse of important bridges due to extreme tsunami, it is recommended that sufficient clearance for wave height of tsunami is ensured for bridge superstructures. For mitigation of the effect of tsunami, considerations in structural design to mitigate the tsunami force to bridge superstructure, and preparation of a recovery plan, which is also effective to mitigate the effect of tsunami, are recommended.

4.3 Revision of Ductility Design Method of Reinforced Concrete Bridge Columns

To improve the accuracy of evaluation of ductility capacity of reinforced concrete bridge columns, limit states of reinforced concrete bridge column are redefined considering required seismic performance of bridges, nonlinear cyclic behavior and damage progress of reinforced concrete bridge columns, and a new evaluation method of ductility capacity including a new equation that estimates plastic hinge length and allowable tensile strain of longitudinal reinforcement, which determines limit state of reinforced concrete bridge column, is proposed considering buckling behavior of longitudinal reinforcement [6].

Table 1 summarizes the seismic performance of bridges, and the proposed definition of limit states of reinforced concrete bridge columns. The damage condition at each seismic performance level (called “SPL” in the following text) is also shown in the table.

The SPL 2 requires that bridges sustain limited damages after an earthquake and are capable of functional recovery in short period, which means damage of structural members is limited and the structural members sustain its capacity of lateral force and energy absorption. Based on these requirements, the limit state of reinforced concrete bridge columns at the SPL 2 is defined at the point where significant degradation of energy absorption capacity has not yet been observed and damage is easily repairable in short period because significant damage such as spalling of cover concrete or buckling of longitudinal reinforcement has not yet occur. The limit state at the SPL 3 is defined at the point just before significant degradation of lateral force capacity is observed, which is the definition same as the method specified in the 2002 specifications (hereinafter referred to as the conventional method).

In this revision, a new equation estimating the plastic hinge length L_p (Eq. (1)), and those estimating tensile allowable strain for the repairable limit state, ε_{st2}, and the ultimate limit state, ε_{st3}, were introduced (Eq. (3) and

(4)).

$$L_p = 9.5\sigma_{sy}^{1/6}\beta_n^{-1/3}\phi \qquad (1)$$

$$\beta_n = \beta_s + \beta_c \qquad (2)$$

$$\varepsilon_{st2} = 0.025L_p^{0.15}\phi^{-0.15}\beta_s^{0.2}\beta_c^{0.22} \qquad (3)$$

$$\varepsilon_{st3} = 0.035L_p^{0.15}\phi^{-0.15}\beta_s^{0.2}\beta_c^{0.22} \qquad (4)$$

where σ_{sy} is the yield strength of longitudinal reinforcement, ϕ is the diameter of longitudinal reinforcement, β_s is the stiffness of the spring that represents restraint of transverse reinforcement, and β_c is the stiffness of the spring that represents restraint of cover concrete.

Using the plastic hinge length given by Eq. (1) and the allowable tensile strain given by Eqs. (3) and (4), the displacement at each limit state was computed, and compared to the test results. Figure 9 shows the relation of lateral displacement at the ultimate limit state obtained from the cyclic loading tests and from the computation. The accuracy on evaluation of ultimate ductility of reinforced concrete bridge column is improved from 36.5% to 17.5% by using the proposed method.

4.4 Applicability of Reinforced Concrete Bridge Columns with Hollow Sections for Plastic Hinge Region

Reinforced concrete bridge columns with hollow sections have been used for tall bridge columns constructed in mountain area in order to reduce the self weight of bridge column, and to reduce the inertia force induced in its foundation. Based on the cyclic loading test results for columns with hollow sections conducted after the Kobe earthquake, the same ductility design method to the solid section has been used in seismic design. However, the structural conditions have been changed over 15 years. For example, the wall thickness has become thinner, the axial stress has become larger, and the amount of longitudinal reinforcement has become larger, which generally result in smaller ductility capacity and severe damage.

To evaluate the effects of such structural change, a series of cyclic loading tests have been conducted at PWRI [7]. It is found from the tests that the conditions listed above causes severe damage in the compression flange and also severe damage at the inside wall as shown in Figure 10. Besides, it is not easy to inspect the damage of the inside wall after an extreme earthquake, and a method has not yet been available to evaluate the damage of the inside wall from the damage of the outside wall.

Based on these results and considerations, it is recommended in the specifications as shown in Figure 11 that a hollow section shall not be used in the plastic hinge region, and haunches shall be provided at four corners inside the hollow sections and at region around the end of hollow section to prevent severe damage.

4.5 Introducing of Design and Construction Principals of Approach Embankment

The damage of main structural members of bridges caused by the recent major earthquakes has been decreased. This is because the newly bridges were designed by the upgraded design specification and seismic retrofit of the existing bridges, which were retrofitted the piers and installed unseating prevention systems, have been progressed. On the other hand, difference in level between abutment and backfill soil, and damage of pier beam by applied for inertia force of superstructure though bearings or unseating prevention devices became remarkable as critical causes of emergency operation after the earthquakes. It is easy to repair the difference in level of road surface in comparison with the other structural members. However, lessons learned from the 2011 Great East Japan Earthquake, they need a lot of time to repair in case that a lot of damages would occur in wide area at once even if each damage might be small. Therefore, it is newly prescribed the approach embankment whose part should be designed and constructed to keep the continuity of the road surface between the bridge and the embankment adjacent to the abutment. The approach embankment should be constructed using soil material that compacts well and ensures sufficient stability and drainage.

5. NEWLY INTRODUCED POINTS BASED ON THE RECENT RESEARCHS

5.1 Introducing of Design and Construction about Integral Abutment Bridge

To reduce the total investment cost of road bridges, it is important to reduce the maintenance costs in addition to the initial costs. One reason contributing to high maintenance costs is damage to bearings at the abutment and the expansion joints. Particularly, the ratio of costs for bearings and expansion joints relative to the total cost of the road bridges is high for short and medium class bridges. To resolve these problems, it is effective to introduce the integral abutment structures which can omit bearings and expansion joints.

Integral abutment bridges are not widespread in Japan although this type of bridge was first introduced experimentally about 20 years ago. The reason is the serious maintenance challenges such as the cracks in the pavement between the abutment and the approach embankment, and the lack of adoption of systematic design methods. Moreover, seismic design of the structure is also a key factor in Japan, just as it is in some U.S. states. However, it was not clear in earlier periods whether the seismic performance verification methods for integral abutment bridges were appropriately executed, especially for extreme earthquake events such as Level 2 earthquakes. Against this background, in 2006, PWRI commenced research of design and construction methods for integral abutment bridges as a cooperative program involving four technical associations. This work led to the issue of the new guideline in 2012 [8].

Based on this research, it is newly prescribed about fundamental principal of design and construction of the structure that an abutment jointed to a superstructure rigidly. In addition to an integral abutment, a portal frame bridge is also targeted at this regulation as shown in Figure12. For example, since the backfill is expected to provide resistance, the specifications of the approach embankment of the integral abutment bridge are higher than the other type of abutments in the area and the control standard values for soil compaction. The detail matters about the design and the construction of integral abutment is described in the guideline as mentioned before.

5.2 Fatigue Durability of Steel Bridge

The fatigue cracks to penetrate a deck plate in a weld of closed rib and deck plate have been increased in the existing steel slabs. When this crack progresses, traffic operation function might be decreased due to damage of pavement and a cave-in of the road. According to the damage example investigation, it was reported that this crack occurred in case of 12mm of minimum thickness of deck plate. Additionally, fatigue loading experimental tests using full scaled wheel and FE model analyses were carried out to evaluate fatigue durability by difference of the structural detail. These results showed that it is most effective to make thicker the deck plate. Based on these results, it is improved to normalize that thickness of deck plate where wheel load of heavy vehicle is always loaded is more than 16mm in case of steel deck using closed rib.

5.3 Design of Connection of Composite Structure

For the purpose of reducing the cost and rational design, composite structures which connect between the concrete member and the steel member such as corrugated steel plate or steel truss member gradually increase in recent years. It is necessary to assume that the connection of the composite structure have enough durability during an in-service period. However, damage examples due to the corrosion were reported at the connection between the concrete member and the steel member. Therefore, the fundamental requirements of design at the connection of composite structure to secure safety and durability were prescribed. Particularly, it is important that water does not stagnant by establishing a cross grade or the draining off aperture, and appropriate rust prevention, waterproofing at an interface and the embedding part of connection between concrete and steel members. Furthermore, it is also important to design the bridge in consideration of maintenance such as easily and securely inspection in service life.

5.4 Introducing High Strength Steel Rebar

A bar arrangement of RC members, especially the substructure, tends to become overcrowded by strengthening the seismic performance of the bridge after the 1995 Hyogo-ken Nanbu earthquake. As a results, construction quality might be deteriorated by the arrangement of the rebar becoming difficult. As for a head of steel pipe pile, reinforcing rebar is installed to connect the pile to footing rigidly. The reinforcing rebar is connected by welding outside of the pipe in case that strength is insufficient only with reinforcing rebar inside the pipe. However, the construction environment of welding is not good as shown in Figure 13.

To improve these problems, it is effective to use high strength steel rebar. Experimental studies such as cyclic loading tests of pier, pile foundation, and investigation about structural details such as bending radius, were carried out. Furthermore, it become easy to obtain high strength rebar in comparison with the past. Based on these results and background, the upper limit of the yield strength of steel rebar as a normal use was improved from 345N/mm^2 (SD345) to 490N/mm^2 (SD490). This effect contributes to the improvement of the bending strength and ductility. Moreover, welding work at a head of steel pipe pile was not necessary by using high strength rebar as shown in Figure 14. However, it does not contribute to shear strength because performance verification for shear is not enough based on the truss method.

6. SUMMARIES

This paper presents the outline of revisions of the Japanese design specifications for highway bridges. Main topics of this revision are as follows,

- Enhancement about designing the bridge in consideration of the maintenance and redundancy
- Seismic Issues such as revision of design earthquake ground motions and tsunami
- Introduction or improvement of specifications based on recent research results
-

We continue to examine to introduce the load and resistance factor design concept.

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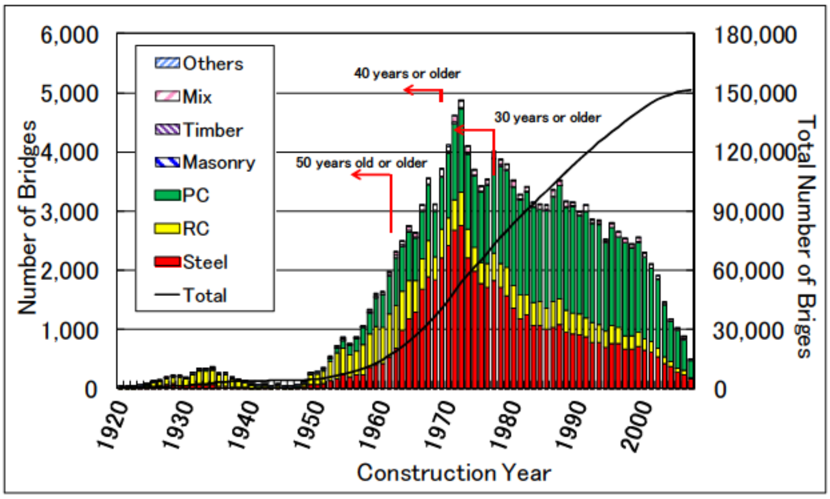


Figure 1 The Number of Road Bridges Constructed in Past Years

Figure 2 Walkway for Inspection

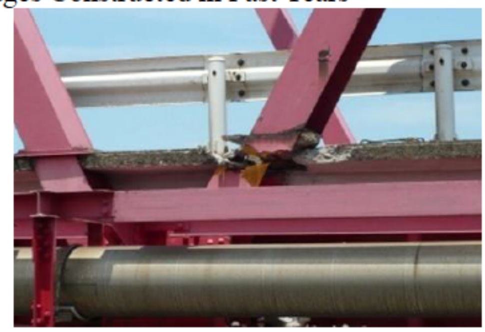
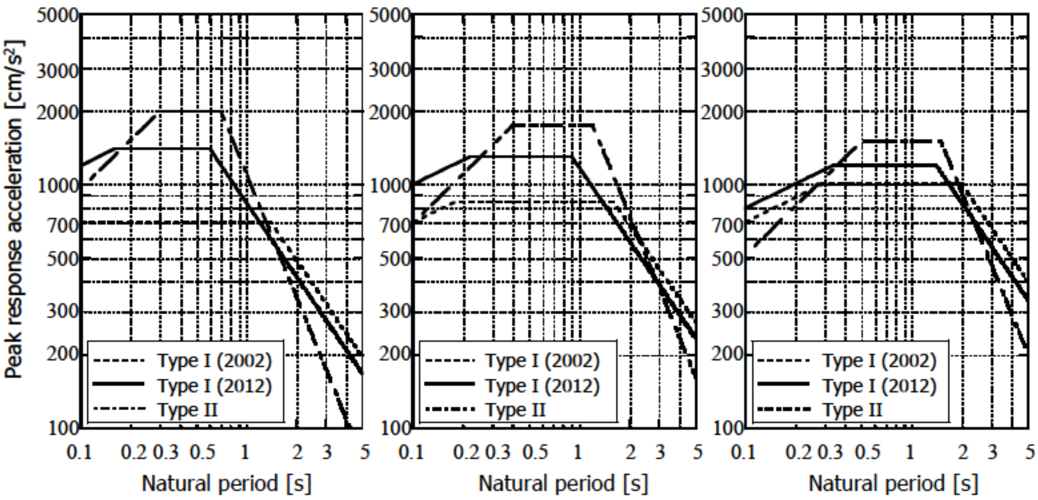


Figure3 Fracture of Diagonal Bridge Bracing in Steel Truss Bridges



(a) Soil profile type I (b) Soil profile type II (c) Soil profile type III

Figure 4 Comparison of standard acceleration response spectra for Level 2 earthquake motions.

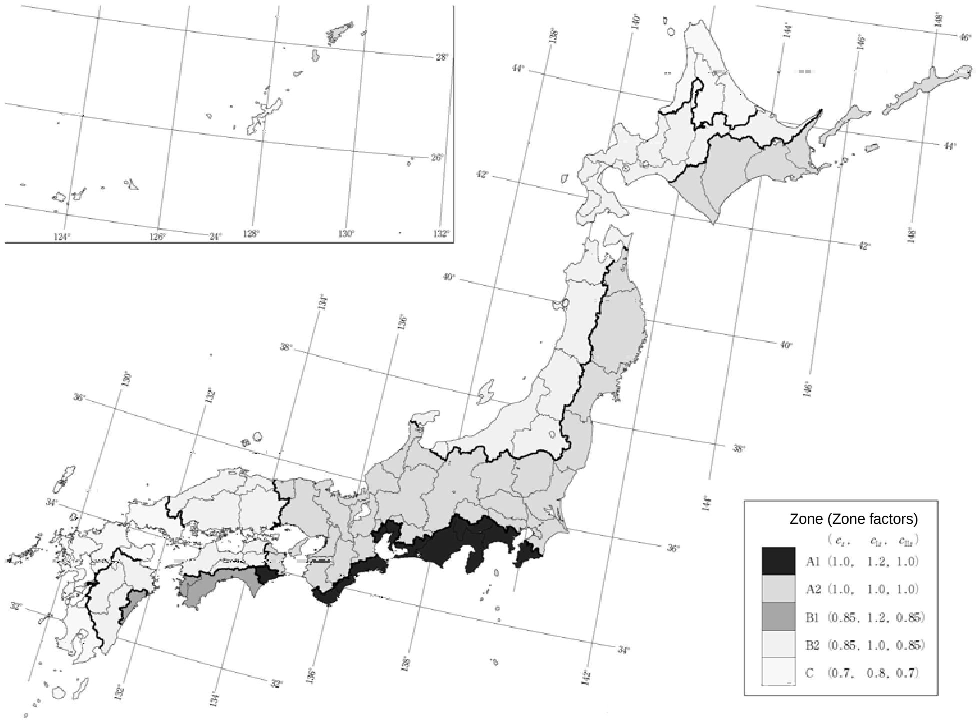
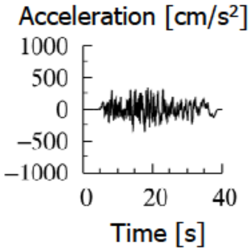
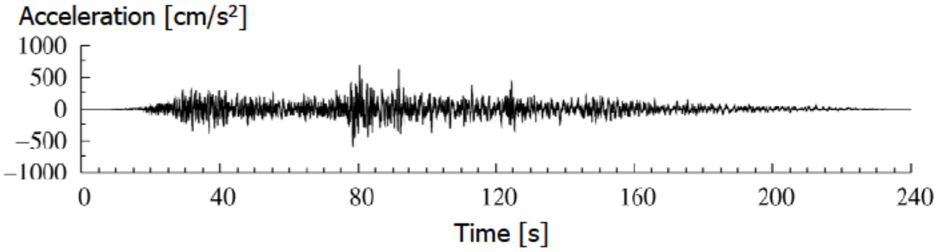


Figure 5 Regional classification for zone factors

Figure 6 Source regions of major plate boundary earthquakes that are taken into account in the revision of the zone factor c_{Iz}

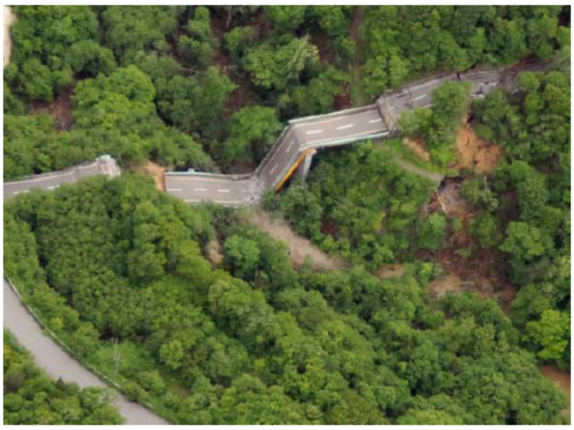


(a) Before revision



(b) After revision

Figure 7 Comparison of acceleration waveforms prepared for time history response analysis. (These examples correspond to soil profile type II.)



(a) Collapse of bridge due to land slide

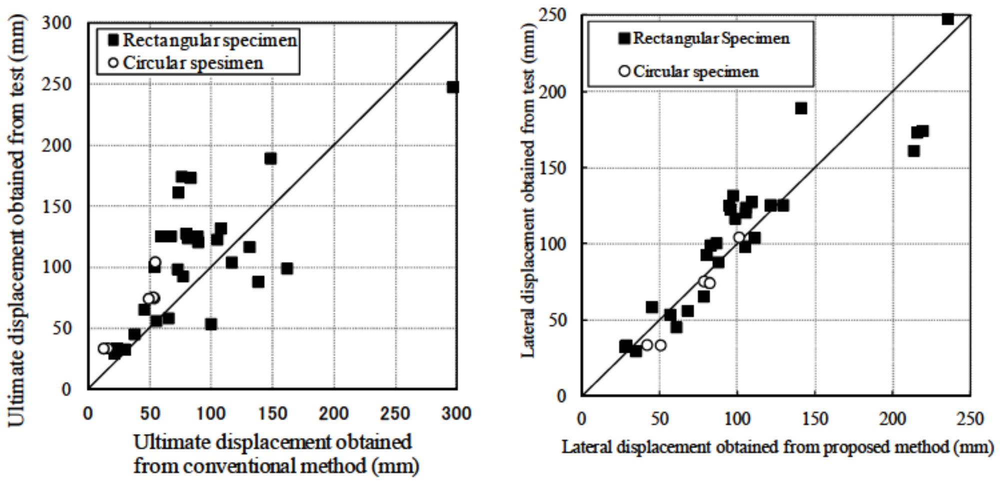
(b) Collapse of bridge due to tsunami

Figure 8 Collapse of bridge caused by effect that are not mainly affected by ground shaking

Table 1 Seismic performance of bridge and limit states of reinforced concrete bridge column

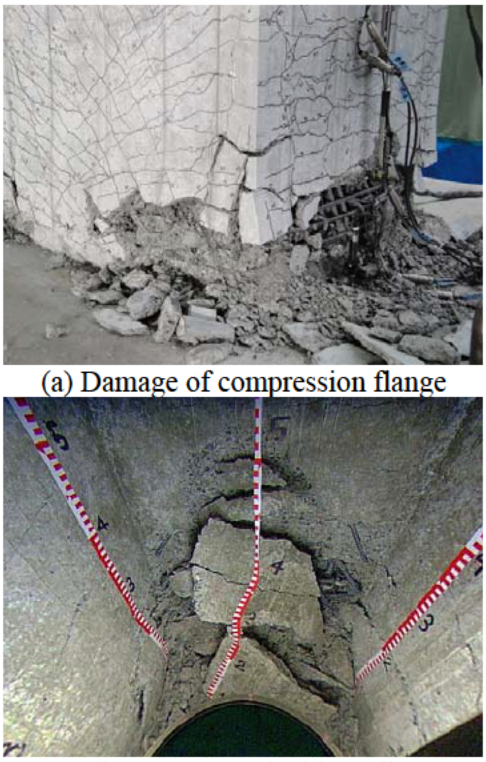
	SPL 2	SPL3
Seismic performance of bridge	Limited damage for function recovery in short period	Prevent fatal damage such as unseating of superstructure/ collapse of column
Limit state of bridge	**Repairable limit state**	**Ultimate limit state**
Limit state of reinforced concrete bridge column	State within a range of easy functional recovery and no significant degradation of energy absorption and lateral force capacity (Condition of 2) in Fig. 2)	State just before significant degradation of lateral force capacity occurs (Condition of 3) in Fig. 2)
Damage condition of reinforced concrete bridge column	Residual flexural cracks or minor spalling of cover concrete	Condition just before buckling of longitudinal reinforcement becomes noticeable after spalling of cover concrete

*) SPL1: Fully operational is required. Limit state of bridge is **serviceability limit state**.
Negligible structural damage and non-structural damage are allowed.



(a) Conventional method (b) Proposed method

Figure 9 Accuracy of estimation of lateral displacement at limit state for ultimate limit state



(a) Damage of compression flange

(b) Damage at inside wall

Figure 10 Damage of reinforced concrete bridge column specimen with hollow section under high axial stress and high longitudinal reinforcement ratio

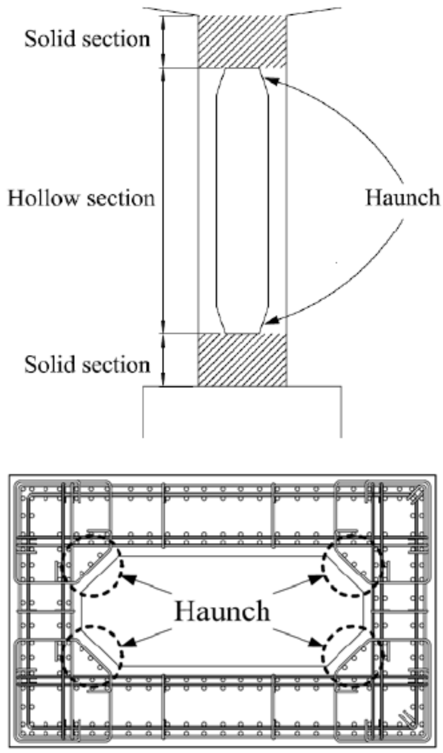
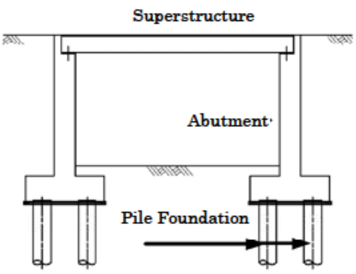


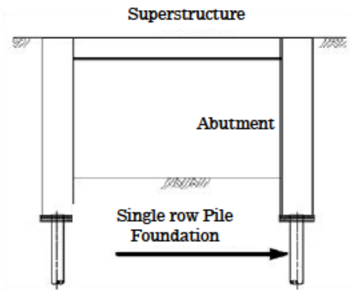
Figure 11 Design recommendations for reinforced concrete bridge columns with hollow section

Figure 12 Comparison of bridge structural characteristics

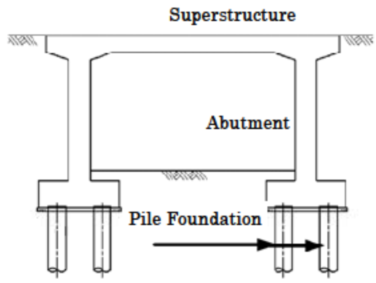
Structural type	a) Conventional bridge	Jointless Structure	
		b) Integral abutment	c) Portal frame
Bearing support	Install	Uninstalled (rigid frame)	Uninstalled (rigid frame)
Expansion joint	Install	Uninstalled (omit)	Uninstalled (omit)
Girder adjustment of expansion from thermal changes	Deform by the foundation	Deform by the flexible pile foundation	Resist by the rigidity of backwall and foundation



a) Conventional bridge



b) Integral abutment bridge



c)Portal Frame bridge

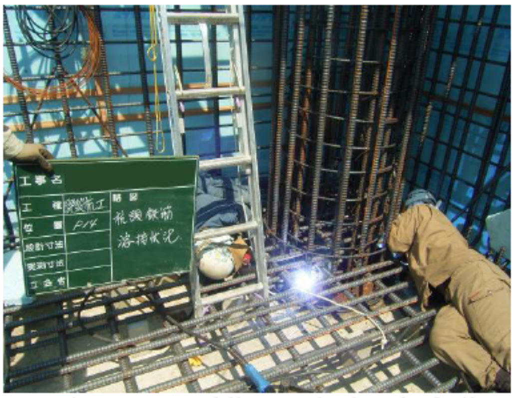


Figure 13 Welding at the head of pile

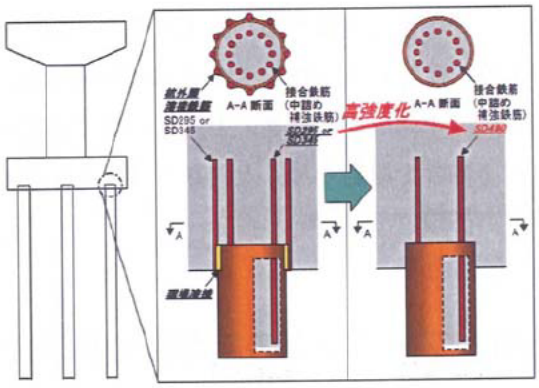


Figure 14 Improvement of Structural Detail at the Head of pile

Tsunami Resilient Ports on the Basis of Lessons from the 2011 Great East Japan Earthquake and Tsunami

by

Takashi Tomita[1], Taro Arikawa[2] and Shigeo Takahashi[3]

ABSTRACT

The 2011 Great East Japan Earthquake and Tsunami caused devastation in coastal areas and ports. Devastation of ports caused stagnation of logistics and industrial operations, resulting in harmful influence on people's lives and industrial activities in the aftermath of the disaster. Lessons learned from the 2011 earthquake and tsunami disaster is importance to draw the worst-case scenario of earthquake and tsunami to save human lives essentially, and to determine an adequate level of tsunami to prevent the estimated disasters by the tsunami. In ports, however, tsunami inundation could be caused because they are developed in low lying flat areas and face to the sea. Therefore, disaster management based on the two hazard levels provides to develop tsunami-resilient ports to earthquake and tsunami disasters. For example, a breakwater with durability and redundancy against tsunami impacts provides reduction of tsunami damage in the port area and prompt reopening the port in the aftermath of the disaster. In addition, a new development of technology may be necessary for prompt reopening of the damage port in which tsunami-debris are spread on and sink below the water surface. This paper introduces the concepts to make a port resilient to tsunamis, based on the lessons learned from the 2011 disaster. The detailed discussions will be provided in an Appendix to the PIANC MarCom Report 112 on "Mitigation of Tsunami Disasters in Ports."

KEYWORDS: Tsunami, Port, the 2011 off the Pacific Coast of Tohoku Region Earthquake, Breakwater, Seawall, Worst-case scenario.

1. INTRODUCTION

The 2011 Great East Japan Earthquake and Tsunami severely damaged coastal areas and ports along the northeastern coast of Honshu Island, Japan. Devastation of ports caused stagnation of logistics and industrial operations, resulting in harmful influence on people's lives and industrial activities in the aftermath of the disaster. Stagnation of logistics caused shortage of various goods in the damaged areas including gasoline and heavy oil which are vital for people's life and prompt recovery works. Stagnation of industrial works caused depression of industrial activities in the damaged areas and national economics, resulting in the loss of employment in the damaged areas. Resiliency of port is the vital essence to secure the national and worldwide distribution networks as well as secure people's life and economic activity.

An important lesson learned from the 2011 disaster, especially the tsunami disaster, is to prevent and mitigate possible disasters based on two tsunami hazard levels. A level (Level 2) is based on the largest-possible mega tsunami and the other level (Level 1) is based on the tsunami occurring more frequently than the Level 2 tsunami and causing major damage despite its relatively lower tsunami height. Against the Level 2 tsunami, the aim of the disaster management is to prevent the loss of lives. To achieve this, we should develop integrated measures consisting of not only structures to reduce tsunami impacts but also a land use plan and system to support people's evacuation [1]. Against the Level 1 tsunami, the fundamental measure is to prevent tsunami inundation with structures such as breakwaters and seawalls. However, in port areas that industrial and logistic

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facilities exist in the low-lying areas, countermeasures should be prepared to support evacuation of workers and visitors in the ports including offshore tsunami observation [1].

This paper introduces that a port is resilient to tsunamis, based on the lessons learned from the 2011 disaster. The detailed discussions will be provided in an Appendix to the PIANC MarCom Report 112 on “Mitigation of Tsunami Disasters in Ports,” whose title is “Tsunami Disaster in Ports due to the Great East Japan Earthquake.”

2. GREAT EAST JAPAN EARTHQUAKE AND TSUNAMI

2.1 Earthquake

The 2011 Great East Japan Earthquake with moment magnitude (Mw) 9.0 occurred at around 14:46 on 11 March 2011, JST, with the hypocenter located at 38°6.2′ N, 142°51.6′ E and 24 km deep [2]. It was the largest earthquake in the history of earthquake observation in Japan, which was a thrust earthquake with the compression axis in the WNW-ESE direction and a typical plate-boundary earthquake between the Pacific and the North American plates. The earthquake ruptured a huge area extending from off Iwate through off Ibaraki prefectures [2]. Such a gigantic earthquake was not hypothesized in the region before the earthquake [3]. Seismic intensity 7 in the JMA scale was observed in Kurihara City, Miyagi Prefecture and seismic intensity 6+ was observed in Miyagi, Fukushima, Ibaraki and Tochigi prefectures [2].

2.2 Tsunami

The sea bottom deformation due to the rupture of fault caused a high tsunami. The maximum inundation and runup heights above the sea surface at the time of tsunami arrival were measured 33.0 m in Sinami-Santiku Town of Miyagi Prefecture and 40.0 m in Ryori of Ofunato City, Iwate Prefecture [4].

The tsunami was also observed in the offshore sea with the GPS-mounted buoys [5], which were moored at depths of 100-400 m at a distance of 10-20 km from the shoreline. Figure 2.1 indicates the observed tsunami profiles off

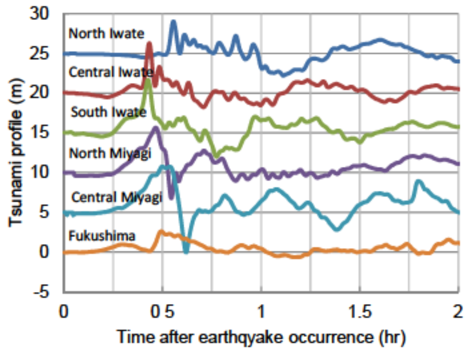


Figure 2.1 Tsunami profiles observed with GPS-mounted buoys

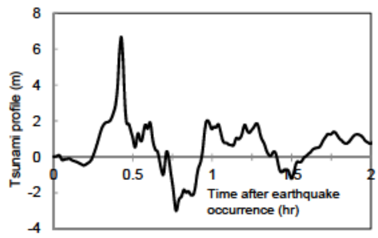


Figure 2.2 Tsunami profile observed with South Iwate GPS-mounted buoy

Photo 2.1 Tsunami overflowing the breakwater of Kamaishi Bay (Photo courtesy of the Japan Coast Guard)

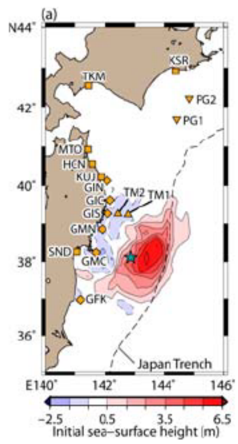


Figure 2.3 Tsunami source estimated with the observed tsunami profiles off the coasts of Tokoku and Hokkaido regions on the assumption of the rupture velocity of 2 km/s.

the coast of Tohoku Region, and Figure 2.2 is also the tsunami profile within two hours after the earthquake occurrence which was observed with the GPS-mounted buoy of South Iwate off Kamaishi Bay. These profiles include apparent sea level rise which reflects ground subsidence in the base stations on land that give reference altitude to calculate vertical displacement of the buoy accurately. The observed tsunami height off Kamaishi Bay is 6.1 m after correction of the ground subsidence. The tsunami is considerably high in the deep water area of 200 m. If this high tsunami propagates on a plane beach, the tsunami increases to 11.0 m in height in a shallow water area of 20 m, following Green's Law which is derived from the energy conservation. Indeed, the tsunami overflowed the breakwater installed at the mouth of Kamaishi Bay, and its maximum height exceeded 11.8 m which was estimated with the analysis of Photo 2.1 [6]. Note that the tsunami source was inversely estimated as shown in Figure 2.3 [7], using the tsunami profiles observed with the GPS-mounted buoys and seabed pressure gauges in the open sea, which is good agreement with physical explanation of the fault rupture.

3. TSUNAMI DAMAGE IN PORT

3.1 Overview

The tsunami caused devastating damage to the coasts of Iwate, Miyagi, and Fukushima prefectures especially. Although various measures have been incorporated to mitigate the disasters using past tsunami data in the areas, the 2011 tsunami destroyed tsunami breakwaters and seawalls that were built to mitigate tsunami impact, because it was several times higher than the tsunami considered in structural design. Further, the tsunami inundation over 2 m in depth caused complete destruction in many wooden houses behind port areas.

In Kamaishi Port, which suffered severe damage from the 1933 Showa Sanriku Tsunami and the 1960 Chilean Tsunami, a tsunami breakwater was built to reduce the maximum tsunami inundation depth to less than 0.5 m, based on the figures of the highest-recorded tsunami in the area of the 1896 Meiji Sanriku Earthquake of Mw 8.5. The design tsunami height was 5.0 m and the considered difference in the water level between the two sides of the breakwater was 2.8 m. The 2011 tsunami that hit the breakwater was over 11.8 m high and damaged the breakwater. However, post-tsunami field surveys to measure the tsunami trace heights and numerical simulations of propagation and inundation of the tsunami have shown that the damaged breakwater reduced the tsunami height in the area behind it [6]. Ordinal breakwaters designed to bring about sea calmness for loading and unloading of cargos were also damaged by the tsunami because they were not designed to withstand impacts as strong as that of the tsunami. However, some breakwaters did reduce the tsunami's impact.

The destruction caused by the tsunami was severe even though it was reduced by the breakwaters. It destroyed seawalls, warehouses, electric devices, and other land facilities in the ports. Debris from destroyed houses and cars flowed inland and to sea with the tsunami's force. Containers and oil tanks were displaced by the tsunami. Some oil was leaked from the damaged oil tanks. Many ships and boats were also displaced. Some vessels were displaced with remaining cargo handling equipment such as buckets in their bodies, which were torn off the loading machines, because the earthquake cut off the electric power supply during the loading and unloading activities. The cargo handling equipment was damaged by the collision of tsunami debris such as the tsunami-displaced vessels as well as the tsunami wave force.

The tsunami also changed the seabed profile locally around the opening section of the breakwater and the wharf. The seabed deformation around the breakwater and wharf resulted in the collapse of their caissons.

Further, the compounded damage also occurred as a result of the action of ground motion and the subsequent tsunami.

3.2 Damage of Breakwater

Breakwaters at the mouth of Kamaishi Bay where the maximum water depth was 63 m, for example, were damaged by the 2011 Tohoku tsunami, as shown in Figure 3.1 and Photo 3.1. The North Breakwater of them (990 m long) consisted of trapezoidal caissons (Figure 3.2) on the foundation mound from -60 m to -27 m in depth in the deep water section. The weight of the caissons was about 36,000 tons. In the

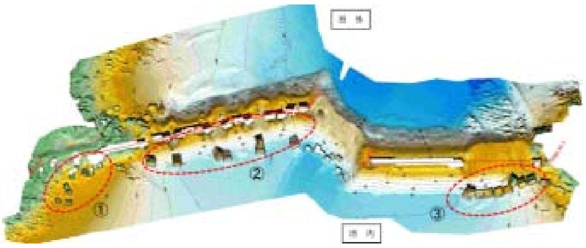


Figure 3.1 Tsunami-distributed caissons of breakwater in Kamaishi Port, (1): Shallow water section of the North Breakwater, (2): Deep water section of the North Breakwater, and (3): Shalow water section of the South Breakwater

Photo 3.1 Damaged North Breakwater in Kamaishi Port

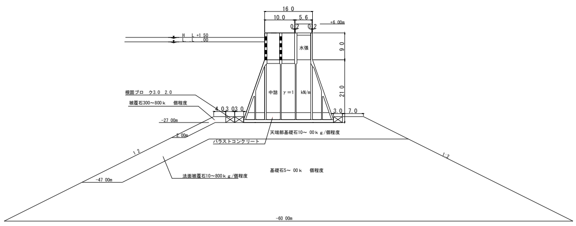


Figure 3.2 Standard cross section in the deep water section of the North Breakwater

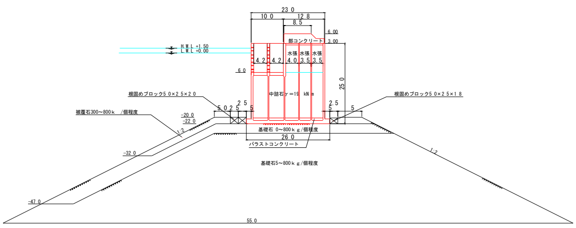


Figure 3.3 Standard cross section in the deep water section of the South Breakwater

shallow section of the North Breakwater, rectangular caissons 10-15 m high were installed. In the South Breakwater 670 m long, three caissons were the trapezoidal caissons and the rest are rectangular caissons of about 32,000 ton (Figure 3.3). Their crest height of the breakwaters was D.L. +6.0 m (T.P. +5.14 m). Along an opening section 330 m long between the North and South Breakwaters, a submerged breakwater was installed, consisting of reinforcement blocks whose crown height was D.L. +19.0 m.

According to the Tohoku Regional Development Bureau of MLIT, seven caissons were slid, 14 were leaned and 1 was undamaged among totally 22 caissons in the deep section of the North Breakwater. While in the shallow section of the North breakwater, 11 caissons were knocked out, five were leaned and six were undamaged. At the South Breakwater, eight caissons were slid, a caisson was leaned and 10 were undamaged in the deep section, while in the shallow part, two were slid and one was leaned and six were undamaged in the shallow section. Five caissons among the undamaged ones were on the mats increasing friction. In the submerged breakwater, 12 caissons were slid and 1 remained.

As shown in Figure 3.4, for example, the foundation mounds in both the North and South Breakwaters were not seriously deformed and the caissons were slid on the mound. This suggests that the caissons could have been gently pulled back inside the bay mainly by the tsunami forces. This suggestion is supported by hydraulic model experiments [8].

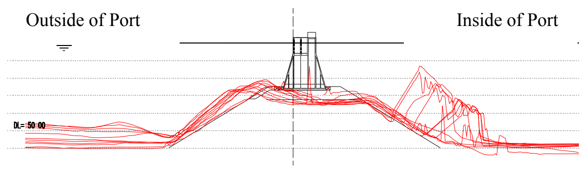


Figure 3.4 Cross section of damage to the North Breakwater

3.3 Damage of Seawall

Damage of seawalls in ports and harbors is breaches of the seawalls were found as well as scattering of the parapet, cracks on surfaces of the seawalls, local damage due to collision of tsunami-debris, breakage along joint surfaces of the seawalls [10]. Foundation scouring in front and behind the seawalls was also found in almost all of the damaged seawalls, as shown Photo 3.2. In addition to impulsive forces due to the run-upping tsunamis with breaking on the land, another important cause to breach the seawalls is the foundation failure due to scouring or drawing-out caused by the repeated action of the incoming and backrush tsunami.

Photo 3.2 Seawalls collapsed toward the landside

4. REDUCTION OF TSUNAMI IMPACTS BY BREAKWATER AND SEAWALL

4.1 Case of Breakwaters in Kamaishi

Numerical simulations of the propagation and inundation of the Great East Japan Tsunami indicated a reduction in the impact of the tsunami by the offshore breakwater in the Port of Kamaishi, as an example. In the numerical simulations, the tsunami numerical model STOC-ML [9] with a hydrostatic assumption was used. To reduce computation time and memory, the nested grid system was applied. The smallest calculation grid of 5 m × 5 m was applied in whole port area where air-born LIDAR topographic data, was used to make topographic structure data, and the nautical charts and a port planning map were used to make bathymetric data. The tsunami source area was calculated with the modified fault parameters of Fujii and Satake's version 1.0 to fit the calculated tsunami height of the first wave to the one observed by the GPS buoy located off the Bay of Kamaishi.

Figures 4.1 a nd 4.2 show the tsunami profiles calculated at the point of the GPS buoy off Kamaishi Bay and the seaward point of the head of the North Breakwater. The black circles in Figure 4.2 indicate the water surface elevation obtained by analyzing photos and video footage. The calculated results are in good agreement

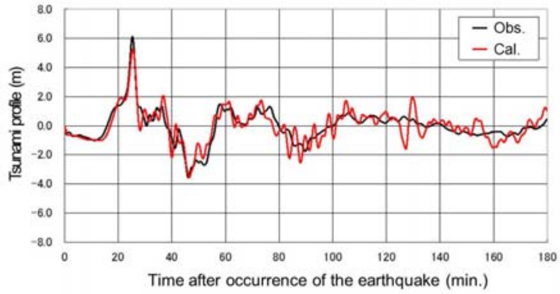


Figure 4.1 Tsunami profiles observed with the GPS-mounted buoy located off Kamaishi Bay (black line) and calculated profile (red line)

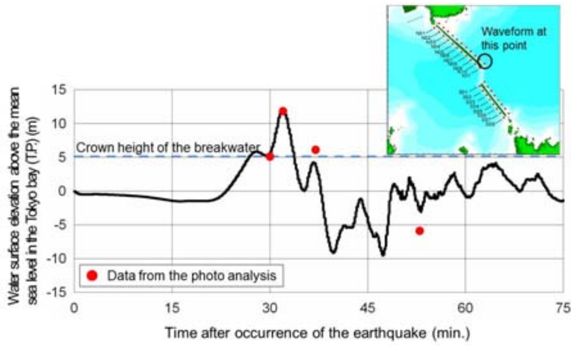
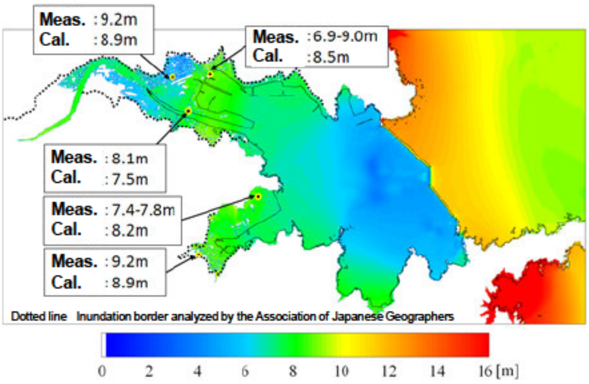


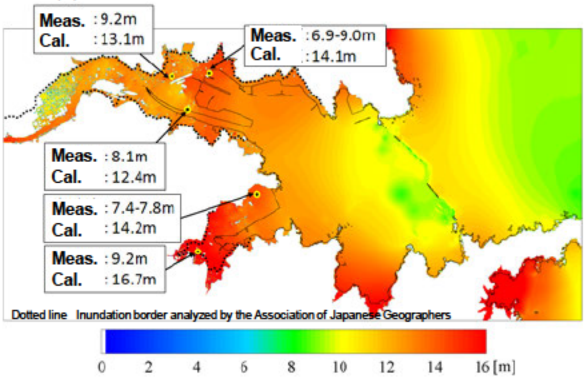
Figure 4.2 Calculated tsunami profile (line) and water surface elevation analyzed from photos (circles) at the head of the North Breakwater in Kamaishi Bay

with the results of the photo analysis. Based on the calculation in the case of no breakwater, the height of the incident tsunami is about 12 m along the north breakwater. This height exceeds the crown of the breakwater by about 7 m, as shown in Photo 2.1 previously. The breakwater was not designed to withstand the force of such a high tsunami, resulting in failure of the breakwater.

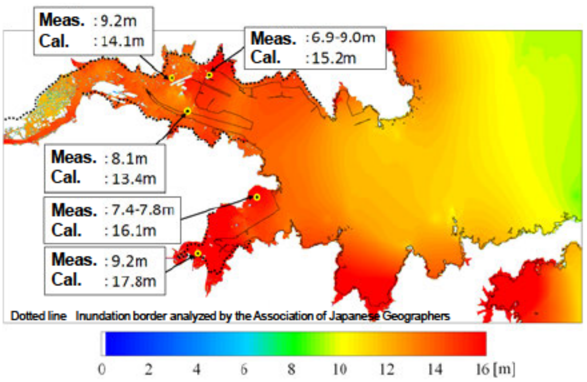
Figure 4.3 shows the spatial distribution of the calculated maximum tsunami height in the sea and inundation height on land with tsunami trace heights measured in the post-tsunami field surveys. In case (a) of the breakwater without failure by the tsunami, the calculated inundation heights are 30-60 cm lower than the measured trace heights. This reduction could be because the breakwater failure that occurred when the first tsunami wave struck the breakwater was not introduced into the calculation. On the other hand, in case (b) where the broken breakwater was set from the beginning of calculation, whose crown height distribution was determined from measurements of the actual damaged breakwater, the calculated inundation heights are 4 m or higher than the measured trace heights. In fact, looking at photo 2.1 that shows the tsunami around the peak of the first positive wave which



(a) Case of the breakwater without failure



(b) Case of the damaged breakwater from the beginning of calculation



(c) Case of no breakwaters

Figure 4.3 Distribution of the maximum tsunami height in the sea and inundation height on land, depending on the situation of the breakwaters

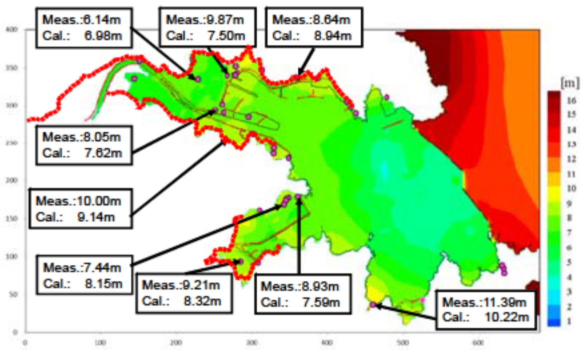
is the highest tsunami wave in Kamaishi, the tsunami overflowing the breakwater has a long crest line with the same height. This indicates that no serious failure of breakwater occurred until this moment. Therefore, the measured tsunami inundation heights lie between those of case (a) and case (b), and comparably near those of case (a). This indicates that the breakwater has probably resisted the force of the tsunami until the peak of the first tsunami wave, resulting in the reduction of tsunami intrusion flux and impact to the port. It should be noted that the damaged breakwater reduced the tsunami inundation height by 40% around the coast inside the port because the calculated inundation height is about 14 m in case (c), which was calculated for the bay without breakwaters, and the measured trace heights are about 8 m. According to the calculations, the inundation areas are 3.38 km2 (the area with inundation depth of 2 m or deeper is 76% of the total inundation area), 4.57 km^2 (90%), and 4.87 km^2 (90%) in the cases of (a), (b), and (c), respectively, confirming that the breakwater reduces the area of inundation.

4.2 Case of Seawalls in Kamaishi

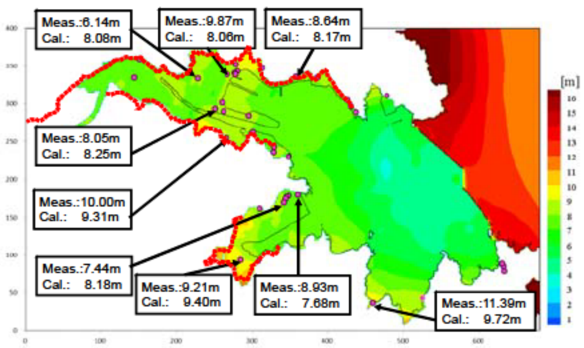
As an example of the reduction of tsunami impact by seawalls, a numerical simulation of the seawalls in Kamaishi Port is presented. Case (a) of the breakwater that did not fail is discussed here because the calculated inundation heights are closest in value to the measured heights in the post-field surveys. In this section, further two conditions of seawalls are compared: case (a-1) of an actual arrangement of 4 m high seawalls along the coastline (red lines in Fig 4.4) and case (a-2) with no seawalls. The calculation area is constructed with nested grids of 5.4 km, 1.8 km, 600 m, 200 m, 100 m, 50 m, 25 m, and 12.5 m.

Figure 4.4 Arrangement of seawalls in Kamaishi Port

Figure 4.5 indicates the spatial distribution of the maximum tsunami height in the sea and inundation height on land as well as the inundation heights measured in the post-field surveys. Both the tsunami height and inundation height of case (a-2) (no seawalls) are several tens of centimeters higher than those of case (a-1) with the actual seawall arrangement. The calculated areas of inundation are 3.26 km^2 (the inundation area with inundation depth of 2 m or deeper: 2.55 km^2) in case (a-1), and 3.33 km^2 (2.62 km^2) in case (a-2). The inundation areas in the case of the actual seawall arrangement are almost the same as those in the case of no seawalls. The reason for this similarity is that the tsunami overflows the seawall at a height of 4 m above the crown height of the seawall which is approximately 4 m from the sea surface at the tsunami arrival time, resulting in a massive amount of seawater intruding from the coast into a narrow low-lying flat area.



(a) Case (a-1) of actual seawall arrangement



(b) Case (a-2) of no seawalls

Figure 4.5 Distribution of the maximum tsunami height in the sea and inundation height on land depending on the seawalls along the coast

In addition, the reduction of the tsunami impact due to the breakwater delays the overflowing of the tsunami over the 4 m high seawall along the coast by about 6 min, on the basis of the calculations, giving people more time to evacuate.

5. TOWARD TSUNAMI-RESILIENT PORTS

This section discusses the following five major items as future countermeasures for tsunami resilient ports.

i) Worst-Case Scenario and Disaster Mitigation
ii) Strengthened Tsunami Defenses
iii) Tsunami Resilient Coastal Towns
iv) Vertical Evacuation, and
v) Tsunami Observation and Warning

5.1 Worst-Case Scenario and Disaster Mitigation

(1) Worst-case scenario and performance design

People living on the east Coast of Japan have been taking necessary precautions against tsunamis since the devastating experiences of past tsunami disasters including the 1986 Maiji-Sanriku Tsunami. However, the 2011 tsunami was immensely destructive and claimed many lives. The major cause of the damage was the incorrect estimation of the size of tsunami. It was significantly larger than that predicted by scientists, more than twice the predicted height on some coasts.

After the disasters by the 2004 Indian Ocean Tsunami and Hurricane Katrina, the importance of preparation for the worst case scenario—a case exceeding ordinary design levels—was pointed out [11]. Table 5.1 shows the measures to be taken in a worst-case scenario under the performance design. The worst case is defined as a Level 2 tsunami assuming an occurrence probability of one every 1000 years, while a Level 1 tsunami is based on a conventional tsunami assuming a probability of one every 50 or 150 years. For the Level 1 tsunami, we aim to prevent the tsunami disaster in order to save lives, property and the economy. For the Level 2 tsunami disaster mitigation is considered; the goal is to save lives, reduce damage to property, and prevent catastrophic damage to ensure early recovery.

Table 5.1 Performance design for tsunami defense

	Design tsunami	Required performance
Level 1 Tsunami	Largest tsunami in modern times (return period: around **100** years)	**Disaster Prevention** • To protect human lives • To protect properties • To protect economic activities
Level 2 Tsunami	One of the largest tsunamis in history (return period: around **1000** years)	**Disaster Mitigation** • To protect human lives • To reduce economic loss, especially by preventing the occurrence of severe secondary disasters and by enabling prompt recovery

(2) Disaster prevention and mitigation

Table 5.1 shows both disaster prevention and mitigation. Disaster prevention is considered for the Level 1 tsunami, while disaster mitigation is considered for the Level 2 tsunami. Disaster prevention refers to designing tsunami defenses to avert tsunami damage. The technology to design and build defensive structures is relatively simple and feasible. The stability of the structure against the predicted tsunami is clear, and therefore the responsibility of the disaster management body is well defined.

Disaster mitigation is not so simple. It is difficult to estimate the extent of failure of coastal defenses and the extent of damage due to rushing currents and inundation. It is more difficult to estimate the secondary damage and economic consequences of the disaster, in addition to administratively and technologically determine the allowable damage level of each town or city. Due to the difficulties in disaster mitigation, the disaster management body of local and central governments is based on disaster prevention. We use the term “disaster mitigation” but not as an administrative term.

It should also be noted that during prevention planning against future tsunamis, the predicted height of the tsunami was kept low due to the high costs involved in tsunami prevention. Even after the 2011 disaster, many people in the local and central government hesitate to consider disaster mitigation in their disaster management scheme. Business continuity planning is a comprehensive scheme for disaster mitigation that is still in its developing stages.

(3) A new worst-case tsunami scenario

Since the summer of 2012, a new worst-case tsunami scenario has been discussed in many organizations including the Disaster Management Council (DMC). As the probability of occurrence of a tsunamigenic earthquake in the Nankai Subduction Zone is high, the worst-case tsunami in the region is being intensively investigated.

Figure 5.1 shows an earthquake model considered by the Huge Earthquake Model Committee in the Disaster Management Council of the Cabinet Office (DMC). This is an Mw 9.1 Earthquake Model in which the old 2003 model is extended to include the Hyuganada Earthquake off the coast of Kyushu and an earthquake along the trough. Figure 5.2 shows the maximum tsunami heights calculated with the newly estimated earthquake model. The bar graph indicates the tsunami heights at ports, which is significantly higher than those of the previous estimation in 2003, in some cases more than twice.

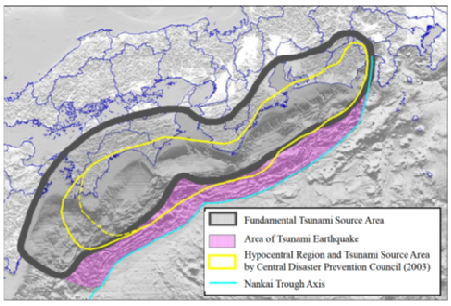


Figure 5.1 New earthquake model in the Nankai Trough

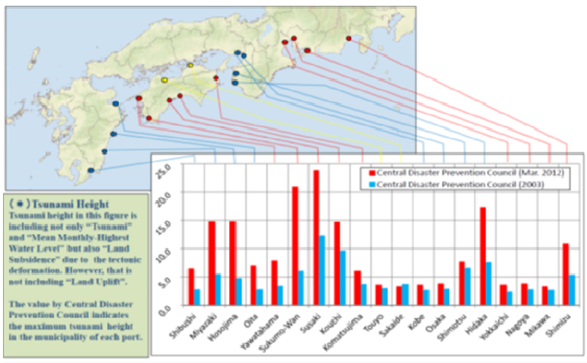


Figure 5.2 Estimated tsunami heights due to earthquakes at Nankai Trough

It is important to consider the ultimate worst case to plan the evacuation model for people in affected areas. It is also essential to design high-priority facilities such as nuclear power plants. However, it is difficult to consider mitigation of disaster against such a huge tsunami, except for evacuation. Administrations consider the Level 1 tsunami for disaster prevention and the Level 2 tsunami only for evacuation. Disaster mitigation against tsunamis in the range between Level 1 and the worst-case tsunami is not considered. Indeed, the occurrence probability of the worst-case tsunami is very low but the occurrence probability of the cases between the two tsunamis cannot be neglected. Tsunami disaster mitigation should be considered for tsunamis between the two levels.

5.2 Strengthen Tsunami Defenses

(1) Disaster mitigation and strength

For the Level 2 tsunami, we can mitigate the disaster by maintaining strong tsunami defenses to reduce the tsunami height. Figure 5.3 demonstrates that the Level 1 tsunami would not exceed the height of the tsunami defense, while the Level 2 tsunami might exceed the height. It is important to have strong, reinforced tsunami defenses that will withstand even the overtopping current of a Level 2 tsunami.

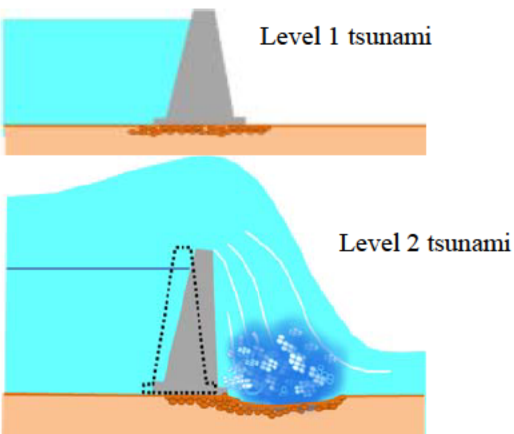


Figure 5.3 Disaster prevention for the Level 1 tsunami and mitigation for the Level 2 tsunami

Figure 5.4 shows the relation between the storm surge level and the damage level reported by the post-disaster survey of Hurricane Katrina [11]. A similar relation can be found in the Great East Japan Earthquake and Tsunami Disaster.

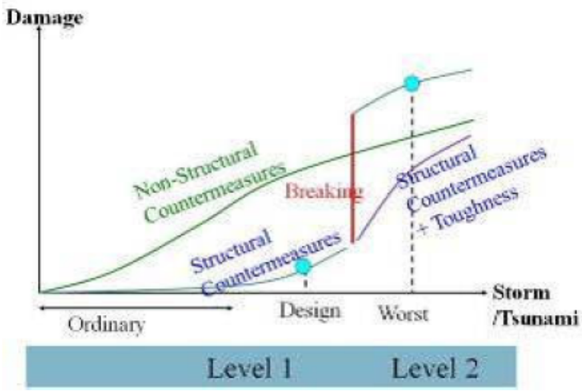


Figure 5.4 Relation of storm surge (tsunami) Level and Damage

Hurricane Katrina

Because many coastal areas in the Gulf of Mexico took no structural countermeasures, the damage increased as the storm surge level rose; hence, the only defense for the population was evacuation. On the other hand, the city of New Orleans had storm surge defense structures. The damage can be minimized due to these structures but the damage increases sharply if the storm surge exceeds the design level, as shown in the figure. In such scenarios, people believe that they are protected by the structures and are reluctant to evacuate, resulting in a high number of casualties. It should be pointed out that the sudden failure of the structure can significantly increase the damage and the robustness of the structures is vital to avoid such sudden destruction, reduce the damage, and avoid casualties. Disaster mitigation can also be expressed as the difference between the damage with structural and non-structural countermeasures.

(2) Strength of the defensive structures

<u>Scouring and Trough-wash Protection</u>

The tsunami defense structure needs to be strong enough to withstand tsunami overtopping currents. The 2011 tsunami resulted in many tsunami defense failures due to scouring and through-wash. Tsunami defense mechanisms can provide maximum protection even during overtopping if appropriate scouring and trough-wash protection is implemented.

<u>Redundancy</u>

Breakwaters and seawalls are generally large structures built to resist huge storm waves; therefore, many of them can withstand the Level 2 tsunami if scouring and trough-wash are prevented. If the structure's main body is not strong enough against the Level 2 tsunami with scour and trough-wash protection, reinforcement of the structure is recommended. This reinforcement is an important measure for added resilience of the tsunami defenses.

Breaking strength

For cost-effective and strong tsunami defense structures, we need to employ a deformation-based design scheme. Figure 5.5 shows the general relation between deformation and strength. Normally, yield strength is used to calculate elastic deformation in the design. A more economical design can be introduced if we use the breaking strength.

Failure mode can be shifted, for example, from concrete wall sliding to foundation failure. It should be noted that the deformation-based design is part of the performance design concept.

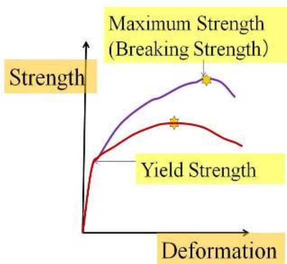


Figure 5.5 Relationship between deformation and strength

5.3 Technology Development to Provide Resilience

(1) Failure mode of protective facilities

Many breakwaters and storm surge barriers have a gravity structure: a concrete body constructed over the foundation ground or riprap. In the gravity structure, a concrete body resists external forces through friction between the bottom of the structure and the foundation. Thus, gravity structures fail in three major ways: sliding caused by the external force exceeding the friction resistance, toppling caused by rotational moment, and bearing capacity failure, which is

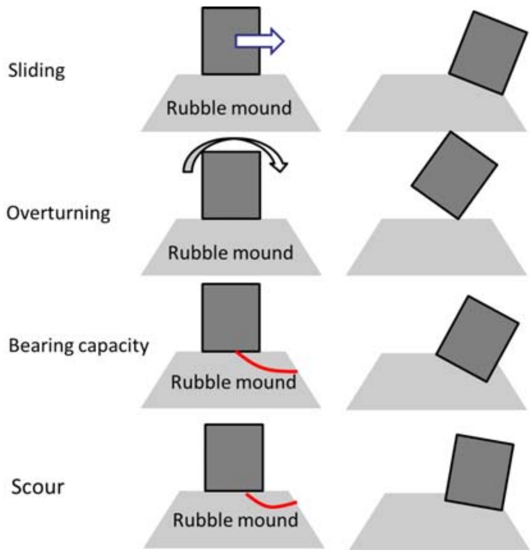


Figure 5.6 Failure modes of breakwaters

the failure of the foundation caused by the body load of the structure. Most of the damage caused by the recent tsunami was due to "scouring" failure mode: scouring of the foundation caused by overflow or joint flow velocity. If the back of the structure is scoured, the foundation is destabilized, causing the body to slide or topple even with a small displacement, resulting in the failure at an external force lower than the external force it was designed to resist. Figure 5.6 shows the categories of failure modes of general breakwaters.

(2) Specific examples of resilient structure

The difference between the extent of the external force of a tsunami and that of storm waves is that a tsunami continues to exert a sustained level of external force over a longer duration. In case of failure, it is likely that the amount of deformation will be large. Thus, when failure by sliding or overturning mode has begun in a gravity structure, the level of deformation is continually impacted by the force of the tsunami and it is difficult to artificially control the scale of deformation. Similarly, scouring depends on the overflow quantity, flow velocity, as well as the duration of action, so it is difficult to control scouring during a tsunami. The duration of the action of a tsunami often includes the length of time until the failure caused by sliding or scouring.

An example of a structure with adequate resistance to tsunamis is the breakwater that is reinforced by widening on its shore-side wall. This is done by placing riprap along the back (shore-side) wall of the structure (Figure 5.7). This widening provides scouring reinforcement behind the structure and increases the body's resistance to sliding and the foundation's bearing capacity. Hydraulic model experiments confirmed that this type of reinforcement increased resistance to both scouring and sliding [8].

With this method, if the widened part is scoured, it is reduced to the failure mode of the original design. Hence, the widening method is designed to increase strength and resist the strong and continuous tsunami forces, gaining time until the failure mode of the breakwater is reached. Another type of reinforcement concerns altering the design of the superstructure so as to increase the distance between the overflow and the back wall of the breakwater.

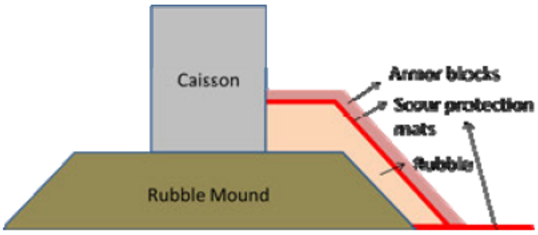


Figure 5.7 Cross section of breakwater with countermeasures against scouring due to overflow of tsunami

To estimate the deformation of foundation and the sliding distance of caisson induced by the tsunami, numerical simulation methods are developing. At least, the tsunami interacting with a structure and its acting force can be calculated with the non-hydrostatic model such as CADMAS-SURF/3D [12], as shown in Figure 5.8.

5.4 Tsunami Resilient Port
Relocation to higher ground is the best solution to survive a tsunami. After the 1896 Meiji-Sanriku tsunami disaster, relocation to higher ground was the only solution. However, technology has advanced since the 19th century and coastal towns have changed significantly.

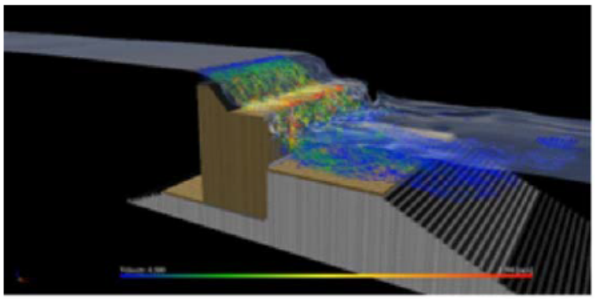


Figure 5.8 Numerical simulation on overflow of the breakwater

Although relocation of towns to higher ground is still an effective solution to save lives, better solutions for saving lives as well as preserving livelihoods should be developed using advanced technology. Because port towns should be located near the sea, we should develop well-planned tsunami-sustainable towns and cities using modern technology. Figure 5.9 shows a compact town with high buildings for residence and commercial use near a coast. By constructing tsunami defense structures with ground reclamations and high buildings, we can create coastal towns that can withstand the Level 2 tsunami.

We have experienced the harshness of the sea, but the sea is a rich place that provides us with abundant resources. To enjoy these benefits, we need to coexist and improve our coastal towns and make them more resilient to natural disasters by using modern technology.

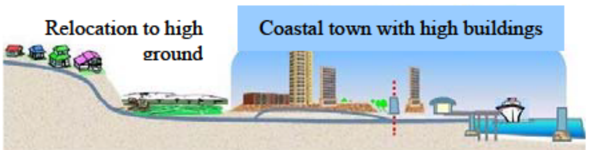


Fig 5.9 Tsunami-resilient coastal town with high buildings

5.5 Vertical Evacuation
A coastal town should be a tsunami-resilient where people can easily and safely evacuate. However, it would be preferable if the people can remain safely in their homes without the need to evacuate during a tsunami. Many lives were lost on roads with traffic jam during the evacuation in the 2011 disaster. Unfortunately, many evacuation centers are quite a distance away

since they are designated as earthquake shelters. It should be noted that earthquake shelters are designed as temporary accommodation for people who lost their homes. On the other hand, tsunami evacuation centers are meant for people to escape from the approaching tsunami. Long-distance evacuation is dangerous for tsunamis because the probability of encountering the tsunami on road is high and can be fatal. It is said that tsunami evacuation should not be horizontal but vertical. It is recommended to escape to higher ground. However, in winter this might mean being exposed to very cold conditions. In addition, some people may not receive hazard warnings. There are many high buildings in coastal towns that can be used as evacuation buildings.

The Disaster Management Council in their post-disaster report stressed the importance of the five-minute evacuation plan using high buildings. Many concrete buildings during the 2011 tsunami remained standing, although they were severely inundated.

6. CONCLUSIONS

1) Devastation of ports caused stagnation of logistics and industrial operations, resulting in harmful influence on people's lives and industrial activities in the aftermath of the disaster induced by the2011 Great East Japan Earthquake and Tsunami. In Japan where economics are concentrated along the coastal areas, disaster mitigation in these areas is a crucial problem.
2) Port areas should be protected to secure the national and worldwide logistic networks as well as secure people's life and economic activity in the damaged areas. To achieve resilient ports, integration of structural and non-structural countermeasures should be considered and developed.
3) A huge tsunami such as the 2011 tsunami causes not only inundation but also destruction houses and infrastructures, generation of a lot of tsunami debris including large vessels, deformation of topography and seabed, and leakage and spreading of oil and danger materials. To manage tsunami disasters, it is necessary to understand and predict such damage induced by the possible tsunamis. To achieve this, development of numerical simulation methods are progressing.
4) Consideration is given to the following items in future tsunami countermeasures; the worst-case scenario and the disaster mitigation for this case, strong tsunami defenses, resilient coastal towns, vertical evacuation using tall evacuation buildings, and improvement in the tsunami observation and tsunami warning system.
5) "Zero Tsunami Casualties" is the primary objective of the tsunami disaster prevention policy, and the crucial countermeasure is the evacuation of people in the affected areas. Marine science and technological developments are essential for establishing more reliable tsunami prediction and warning systems.
6) In the tsunami disaster management, two tsunami hazard levels should be determined to prevent and mitigate possible disasters. The Level 2 tsunami which is the largest-possible mega tsunami is for saving , and the Level 1 which is the tsunami occurring more frequently than the Level 2 tsunami and causing major damage despite its relatively lower tsunami height than the Level 2 tsunami is for planning to prevent tsunami inundation with structural defenses.
7) The damaged breakwaters by the 2011 tsunami had the effect of reduction of tsunami impacts. Thus, a strong or tsunami-resilient breakwater is a measure to reduce the disasters in the port, resulting in resiliency of the port.

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Structural Design Requirement on the Tsunami Evacuation Buildings

by

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ABSTRACT

This paper introduces a structural design method of tsunami evacuation buildings which was discussed after the 2011 Great East Japan earthquake due to the building damage from tsunami and enforced as a notification by Ministry of Land Infrastructure Transport and Tourism.

The ratio of the water depth to the design inundation depth in a hydro static tsunami load, which was 3.0 in the existing design guideline, can be basically selected among 3.0, 2.0 and 1.5 in accordance with the existence of the seaward obstacles and the distance from coasts or rivers. The tsunami wave pressure on the openings such as windows and doors can be negligible, and the design tsunami loads can be simply reduced in proportional with the aerial ratio of the openings on the frame directly tsunami affected. The interim guideline required to be taken into consideration of buoyant forces and impact loads by the debris. The tensile force on individual columns by the buoyancy can be derived from the volume of concrete and residual air in the buildings. The building should be designed to prevent a shear failure of the pile and overturning by the tsunami loads and the buoyant force. In afraid of the local damage on a vertical element by the debris, the residual frame should be able to support the redistributed axial load. The paper finally shows the requirement of base shear coefficient and length of the reinforced concrete building structures for tsunami evacuation buildings in a parametric study of the ratio and the inundation depth.

KEYWORDS: 2011 Great East Japan Earthquake, Buildings, Structural Design, Tsunami Loads

1.INTRODUCTION

The damage from tsunami in the Great East Japan earthquake of 2011 was one of the most disastrous tsunami Japan has ever experienced. A great many lives were lost and many towns were destroyed. For the recovery to proceed quickly, many studies from various points of view are being carried out. In this paper, the results of the examination concerning the structural design method of a tsunami evacuation building, which is included in the tsunami related studies, will be introduced.

Usually, evacuation to a high ground is a basic principle when tsunamis occur. If there is no high ground to evacuate to, a tsunami evacuation building will protect human lives instead of high ground. Therefore, it is required that tsunami evacuation buildings should have a reliable structure which is comparable to a high ground and the safety concerning the evacuation.

The contents of this paper are verifications of validity and examinations of some of the content which require review of structural design methods in "The Guidelines concerning the tsunami evacuation buildings etc. (hereafter, The Guideline [1])," based on the tsunami damage from the Great East Japan earthquake. This examination was carried out as collaboration between the Institute of Industrial Science, the University of Tokyo and the Building Research Institute.

2. A STRUCTURAL DESIGN METHOD FOR TSUNAMI EVACUATION BUILDINGS

Since the Central Disaster Prevention Council issued The General Principles for the Measures

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Table 1. Guideline for Structural Design of Tsunami Evacuation Buildings

1) Not to be collapsed : It has to be confirmed that the tsunami load on each floor of the building will never be higher than the horizontal proof stress. **2) Not to be overturned** : It has to be confirmed that overturning moment by the tsunami load will never be higher than the resistance moment considering buoyancy. **3) Not to be slided** : It has to be confirmed that the horizontal force will never be higher than the friction of the foundation or the horizontal proof stress of the piles. If the resistance against the horizontal displacement of the building can be expected, it can be included in the calculation. - It has to be confirmed that the pressure resistance components in the pressure taking side will not be destroyed by the tsunami wave pressure.

against Tokai Earthquake in May 2003, and The General Principles for the Measures against Tonankai and Nankai Earthquake in December 2003, the necessity of tsunami evacuation buildings, which contributed to the prevention of loss of human lives during tsunami disasters, is receiving more public awareness. Based on this status, the Building Center of Japan (BCJ) examined the structural design method for tsunami evacuation buildings as their independent research project for the 2004 fiscal year [2]. During their research period, the 2004 Indian Ocean Earthquake and Tsunami occurred in December 2004. Under such conditions, The Guideline which the Cabinet Office issued in June 2005, quoted the results of examination by The BCJ in its appendix II entitled "The Basic Way of Considering Structural Conditions."

The collaboration research between the Institute of Industrial Science, the University of Tokyo and the Building Research Institute was carried out as the Building Standards Improvement promotion project No.40 in the 2011 fiscal year "A study of Improvement of Building Standards and others in the tsunami critical areas." In the collaboration research, the structural design method for tsunami evacuation buildings was taken up, which was in the "Guideline," and examined its appropriateness, selected contents which required review, and examined the description, based on the measured inundation depth, elements of buildings, and the damage status of the buildings which experienced tsunami exposure.

The results were reflected in "The interim guideline for the structural conditions of tsunami evacuation buildings etc, based on the building damage from the tsunami in the Great East Japan earthquake," [3]) which was an appendix of the technical advice (MLIT, Housing Bureau, Building Guidance Division No.2570, on Nov. 17, 2011) (we will call this "the interim guideline" and The Notification "Concerning setting the safe structure method for tsunamis which are presumed when tsunami inundation occurs," [4]) (MLIT notification No.1318, on Dec.27, 2011).

2.1 The Design Policy and the Point of Review

In the appendix II of "the Guideline," the Cabinet Office published "the basic way of considering the structural conditions," based on the results of BCJ's independent research. The table of contents reads; 1.1 The range for application, 1.2 Terms to use, 1.3 Structural design, 1.4 Calculation formula of tsunami load, 1.5 Combination of load, 1.6 The design of the tsunami contact surface, 1.7 examination for overturning and sliding. As you see, it is mainly the examination of how not to collapse, not to overturn or not to slide against the tsunami load. Also, it is necessary to confirm that the pressure-resistant components for the tsunami contact side (the side of a building which receives a tsunami load directly) shouldn't lose the resistance capacity against the horizontal force or vertical bearing capacity, and also it shouldn't be destroyed by the wave pressure.

In this review, the way of thinking mentioned above will be followed, as a guideline for structure design in tsunami evacuation buildings. So, concerning the design of tsunami evacuation buildings, an examination for the three conditions which are written in the Table 1 was carried out. Also, concerning the components on the tsunami contact side, they were classified into pressure-resistance components (the ones which shouldn't be destroyed when they received tsunami wave pressure) and non-pressure-resistance components (the ones which are allowed to be destroyed by tsunami wave pressure). And it will be examined if indeed the pressure-resistance components will not be destroyed by tsunami wave pressure.

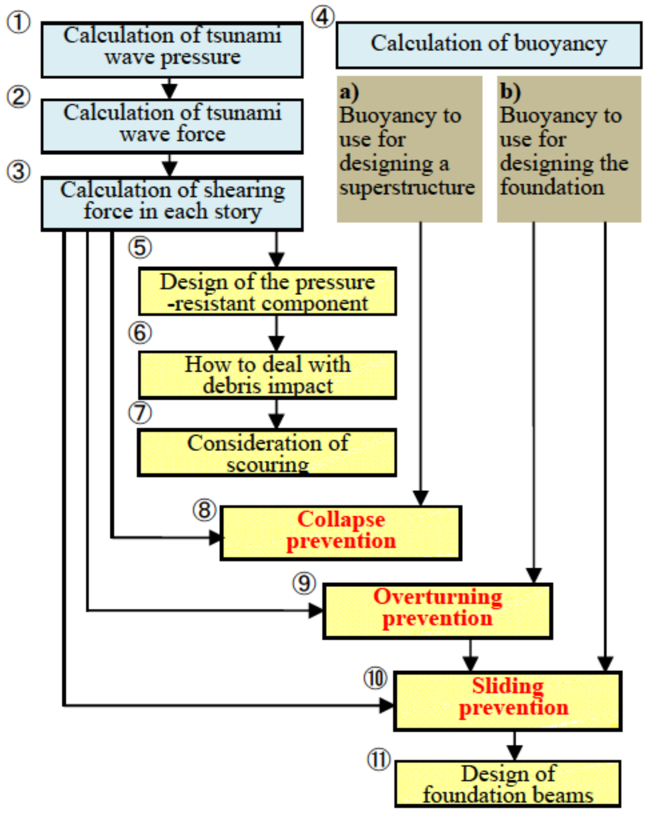


Figure 1. Process of the structural design of a tsunami evacuation building

2.2 Outline of the Structural Design for Tsunami Evacuation Buildings

The process of structural design is indicated in Figure 1. The design for tsunami evacuation buildings will be carried out in the following order. The details of 1), 2), 4) and 6) will be explained in "2.3 Calculation of tsunami wave pressure," "2.4 Calculation of tsunami wave power," "2.5 Calculation of buoyancy," and "2.6 How to deal with debris impact", respectively.

1) Calculation of tsunami wave pressure:
Tsunami wave pressure should be calculated as the static water pressure whose height equals the inundation depth multiplied by water depth coefficient "a."

2) Calculation of tsunami wave force:
Tsunami wave force should be calculated by integrating tsunami wave pressure into the direction of height, considering the decreasing effect of wave pressure because of the openings. Tsunami wave force, which works on each floor of a building, should be calculated by wave pressure from the middle of the height of the floor which is a level below to the middle of the height of the floor concerned, by dividing all the tsunami waver force working on the floor level of each floor.

3) Calculation of shearing force in each story:
The shearing force due to tsunami wave force in each story should be calculated by summing up all the tsunami wave force in all the stories above the concerned floor.

4) Calculation of buoyancy:
There are two kind of buoyancy to calculate.

a) Buoyancy to use for designing a superstructure:
In the examination of collapse, the buoyancy to use for designing a superstructure should be calculated as the sum of the buoyancy for the cubic volume of the structural body under the inundation depth which works on the structure and the buoyancy of air under the floorboard, under the conditions in which a certain amount of water has come into the building from the openings on each floor according to the inundation depth.

b) Buoyancy to use for designing the foundation:
In the examination of overturning and sliding, the buoyancy to use for designing the foundation usually should be calculated by taking the buoyancy for the whole building as working on the bottom of the foundation with an assumption that no water has come into the building.

5) Designing the pressure-resistant component:
It has to be confirmed that columns and walls,

which are pressure-resistant components, should not be destroyed by wave pressure, which works against them. It also has to be confirmed that the bending moment and shear force by wave pressure should never go over the flexural capacity and shear capacity of each of the components, respectively.

6) How to deal with debris impact:
It has to be confirmed that the building will not lose its vertical supporting capacity by debris impact by examining if the axial force supported by a column can be transferred to the neighbor column via the beams attached, assuming the case in which the exterior columns were destroyed by drifting objects.

7) Consideration of scouring:
Prevention measures against scouring, such as having a pile foundation to prevent the superstructure from tilting, or hardening the ground with concrete, should be considered.

8) Collapse prevention:
It has to be confirmed that the horizontal capacity obtained by pushover analysis using tsunami water force obtained by 2) as external force distribution and considering the buoyancy obtained by 4-a), should go over the shear force in each story, which is gained in 3).

9) Overturning prevention:
Each support reaction under the foundation (=axial force of the piles) can be calculated by adding the support reaction by pushover analysis using the tsunami wave power by 2) as external force distribution (no buoyancy included) to the buoyancy which works on each support which can be calculated by dividing the 4-b) buoyancy with the accumulative area. Then, concerning the tension pile, it has to be confirmed that the value is lower than the ultimate tensile capacity (it should be the lesser value in the tensile capacity of the pile or friction of the area surrounding the pile) of the pile. Or concerning the compression pile, the value has to be lower than the ultimate axial bearing capacity of the pile.

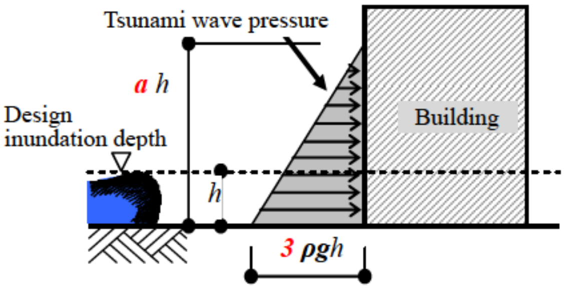
10) Sliding prevention:
It has to be confirmed that the horizontal capacity of the piles according to N-M relationships, here the axial force N is obtained by 9), will go over the tsunami load which works on the pile obtained by 3).

11) Designing the foundation beam:
The foundation beam has to be designed for the force of the superstructure plus the force which is accumulated in the force of the piles.

2.3 Calculation of Tsunami Wave Pressure
The “Guideline,” taking the water depth coefficient “*a*” for 3 as indicated in Figure 2, gave tsunami wave pressure which is three times the inundation depth on a side of the building for design.

This is something that Asakura et. al. [5] suggested concerning the tsunami wave pressure which works on a building by the tsunami which surged over the land beyond the upright seawalls. It was based on the results of their experiments with models in which they had many variations of wave characteristics such as wave height or cycle, slope steepness of the water way and the position of the building, the wave pressure distribution was a triangular distribution, and the maximum height was about three times the inundation depth. This means that this formula for tsunami wave pressure calculation by static water pressure invisibly includes the influence from water velocity. It is considered that some other experiments and suggested formulae can be classified into the “safe” side by the idea mentioned above. Also, Nakano [6] investigated this idea by the data of buildings which had tsunami damage in the 2004 Indian Ocean Earthquake and Tsunami, and it has been proved that they are almost approximate.

In this examination, according to our field survey, we took the number “3,” in the method of tsunami wave pressure calculation in the “Guideline,” for fluctuating numbers because of the power of tsunami. Therefore, we took this “3” for “*a* (water depth coefficient),” and examined this “*a*” based on the actual status of damage. In the examination, the lateral capacity of damaged structures corresponding to the type of failure are estimated firstly, and the water depth coefficient “*a*” is calculated under the assumption that the calculated shearing force at base should be equal to the



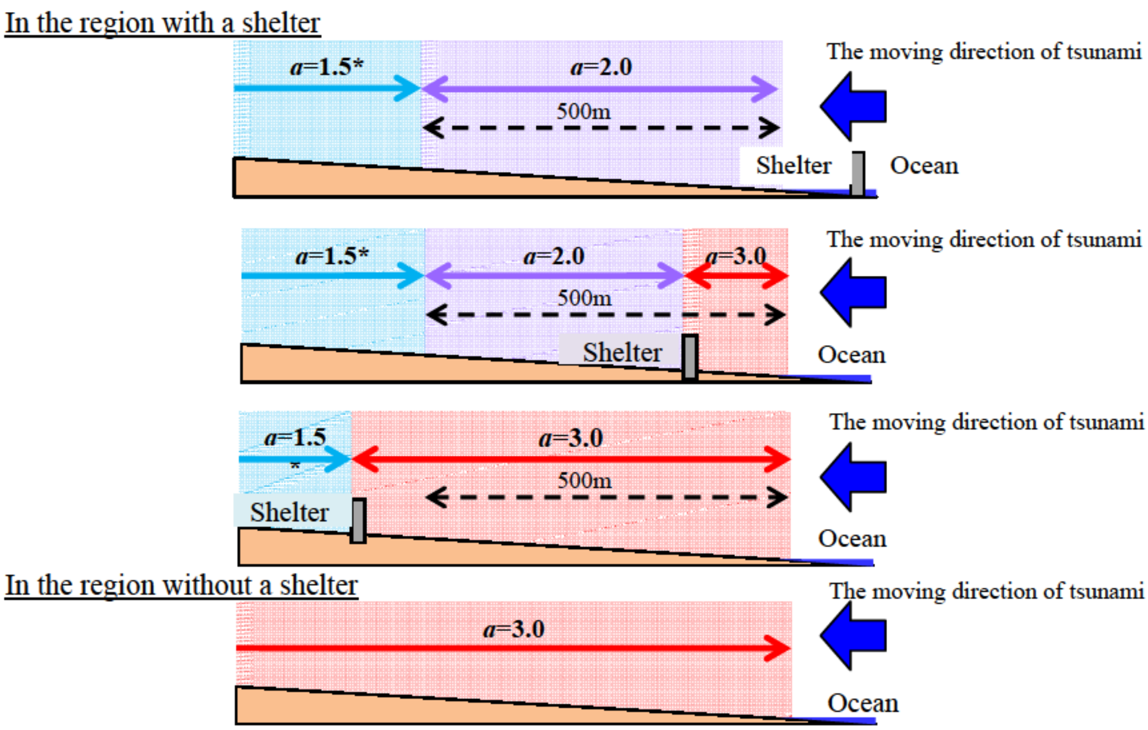
a : Water depth coefficient
h : Design inundation depth (m)
ρ : The mass of unit volume of water (t/m^3)
g : Gravitational acceleration (m/s^2)

According to the "Guidelines," the water depth coefficient "*a*" in the figure is 3, and the hydrostatic pressure which is three times the inundation depth for design is given. But in this examination, "*a*" was investigated based on the actual damage.

Figure 2. Calculation of the tsunami wave pressure

Table 2 Water depth coefficient "*a*"

	With a shelter		No shelter
Distance from seashore or rivers	More than 500m	Less than 500m	Doesn' t matter
Setting the water depth coefficient ***a***	1.5	2	3



**In the region which doesn't have the fluid speed increase, "a" can be reduced into 1.5.*

Figure 3. Relation between the water depth coefficient "a" and the existence of a shelter or distance from seashore and rivers

capacity of the structure when the static water pressure distribution was assumed, by using the measured inundation depth of the site.

Also, as a result of hearing tsunami specialists' opinions, one idea stands out, that since the water power of the tsunami in the Great East Japan earthquake is not necessarily the largest volume we can think of, it is not appropriate to revise the maximum value of tsunami wave pressure only from the damage, under evaluation was indicated. Therefore, it is decided to examine the conditions which can decrease the water depth coefficient "*a*" by taking the existing knowledge "static water pressure which equals three times the inundation depth for designing," for the case whereby the maximum wave pressure works.

The Formula of Tsunami Wave Pressure Calculation: The formula (1) is the tsunami wave pressure calculation which was obtained as the result of the investigation based on the field survey. And Figure 3 is a schema of the water depth coefficient "*a*." Also, the (a) and (b) are the details concerning the water depth coefficient examination.

$$qz = \rho g\,(ah - z) \qquad \text{----------------------------} \quad (1)$$

In this formula,

- ρ : The mass of unit volume of water, 1.0 (t/m^3)
- g : Gravitational acceleration, 9.8 (m/s^2)
- h : Inundation depth for designing (m)
- z : The height of the concerning part from the ground level ($0 \leq z \leq ah$) (m)
- a : Water depth coefficient. If it is confirmed that the Froude number $Fr = u/\sqrt{g\eta}$ is less than 1.0 clearly, "*a*" can be 1.5. ("u"= tsunami velocity, η=inundation depth)

(a) <u>Examination of water depth coefficient from the view of influence of shelters</u>:
The case whereby there are shelters which can reduce tsunami wave power between the building and the tsunami was taken up, as a condition to decrease the water depth coefficient "*a*." In the examination, other structures, seawalls at the mouth of the bay, and tide embankments which are high enough against tsunami (assuming more than half the height of tsunami) are considered as shelters which can be expected to reduce wave power. As a result, it was found out that if there is a shelter which can be expected to reduce tsunami wave power, the water depth coefficient "*a*" will be reduced into 1/1.5 compared with the case without any shelter. Therefore, it can be concluded that if there is a shelter which can be expected to reduce tsunami wave power, the water depth coefficient can be 3/1.5, which equals 2.

Also, since some of the seawalls at the mouth of the bay and tide embankments were destroyed, it is difficult to define the effect specifically. Therefore, facilities and buildings which are in the direction of the tsunami with tsunami evacuation buildings were compared.

(b) <u>Examination of the water depth coefficient from the view of influence of distance from seashore or rivers</u>:
Secondly, the distance from seashore or rivers is taken as a condition to reduce the water depth coefficient "*a*." As a result of the examination, it was confirmed that the power of tsunami (Froude number, Fr) is decreased in proportion to the distance from seashore or rivers. For example, if it is more than 500m away from seashore or rivers, it can be considered that *a*=about 1.0. Since the data from the field survey is limited, the calculation of capacity is a result of simplified calculation, and there is dispersion between the inundation depth from tsunami simulation and the measured inundation depth, it is decided to give the result of the field survey about 1.5 times of margin. Therefore, if there is a shelter and it is more than 500m away from seashore or rivers, the water depth coefficient can be 1.5. Also, it is necessary to check if there is any element to increase the speed of the current, such as positioning the buildings which gather water velocity or down-grade around the building.

2.4 Calculation of Tsunami Wave Force
In this section, the method to calculate tsunami wave power from the tsunami wave pressure which is obtained in 2.3, and the way of treating structural openings for the calculation are explained. From the tsunami wave pressure which is calculated by the method, shear force on each story of the building, support reaction of the building, and the horizontal force which works on

the foundation of the building can be calculated. And by using the method in Table 1, it can be confirmed that collapse, overturning, and sliding will not occur. Also, to confirm that the pressure-resistant components on the pressure receiving side will not be destroyed, the wave power which works on the pressure-resistant components can be calculated by the following method.

1) The formula of tsunami wave force calculation: Tsunami wave force can be calculated in the formula (2), assuming that the tsunami wave pressure in the formula (1) occurs at the same time.

$$Q_z = \rho g \int_{z1}^{z2} (ah - z)\, B dz \quad \text{-----------------} \quad (2)$$

In this formula,

- Q_z : The tsunami force (kN)
- B : The width of the pressure receiving side of the concerning part (m)
- z1 : The minimum height of the pressure receiving side ($0 \le z1 \le z2$) (m)
- z2 : The maximum height of the pressure receiving side ($z1 \le z2 \le ah$) (m)

The tsunami wave pressure which is indicated in the formula (1) expresses tsunami wave force per unit area. Therefore, tsunami wave force can be calculated by integrating tsunami wave pressure for the pressure impact area. The formula (2) is the one tsunami wave force calculation where the minimum height of the pressure receiving side is z1 and the maximum height is z2. Since in some cases the width B is not fixed according to the height, if it is the case, it is recommended to be cautious in integrating it by using the width according to the height of the pressure receiving side.

2) How to treat the openings: When a tsunami works on a building, window glass is destroyed and the wave force which works on the structure will be reduced compared to the case in which the pressure receiving side is made of pressure-resistant components entirely. This means that tsunami wave force can be reduced by openings in the external wall such as windows, doors, and shutters (which is on the pressure receiving side and is made of non-pressure-resistant components). Also, it was found a steel construction which overturned because the exterior wall made of ALC panels survived and received very strong tsunami wave force. Therefore, it should be a principle that the cladding with a steel framework is treated as a part which receives tsunami wave pressure. Also, it is possible to treat them as structures with openings because it may be destroyed at an early stage, but if it is the case, it has to be carefully confirmed that the cladding will be certainly destroyed or come off.

One of the following can be adopted as a method of reducing tsunami wave force because of openings;

- To calculate tsunami wave force by excluding the width of opening parts from the width of the pressure receiving side of each height. (the formula (2)),
- Tsunami wave power multiplied by the area which excludes the opening area from the pressure taking area (= 1- ratio of the opening on the pressure receiving side).

These two methods are compared for the cases when the ratio of opening area changes in the direction of the height. And it is confirmed that the gap between the two methods was within a 10% maximum. Therefore, these two methods can indicate almost the same amount of reduction.

Also, according to the sample which we examined the reduction of wave power by openings with a numerical simulation, the wave force will be reduced more, if the ratio of the opening becomes larger. But, if it is over 30%, since there are interior walls, the reduction of wave force will reach the limit. Based on this result, it can be said that the lower limit of wave power should be about 70% in cases in which there is no opening on the building. Therefore, it is very important to provide routes for water stream and outlet within the building as far as possible [7].

2.5 Calculation of Buoyancy

1) The way of thinking buoyancy: Strong buoyancy works on a building which has small openings. A building which might float in the water was actually found. Since the general

unit weights of steel structure and RC structure are 8 kN/m^2 and 13 kN/m^2 each, the building's own weight will be canceled if there is more than 80cm or 130cm of air layer on each story, respectively. Since not much water can flow in the building which has extremely small openings such as a refrigerated warehouse, the building will be easily floated if the speed of increase of the inundation depth is fast. In most of the overturned building, the ratio of opening on the exterior wall was lower than 0.2.

On the other hand, it is found air gathering spots under the floor slab which was equal in the length to the hanging partition wall even in a building which has a certain amount of openings. Also, by having the inside of the building inundated, the density of the structure will be smaller as much as the density of water (≒1.0). Hence, this much buoyancy has to be considered even in the case if water flowed into the building. Also, if there is a part in which there is no air outlet (ex. Cores), it is better to consider the buoyancy which is equal to its cubic content.

When buoyancy works on a building, the resistance of weight against overturning will be smaller. Also, the friction on the bottom of the foundation against the sliding will be diminished as well. And in the piles and columns of RC structure, since the axial force is reduced, the flexural strength and shear strength will be decreased. In this way, the influence of buoyancy is extremely large.

2) <u>The method to calculate buoyancy</u>:
It is desired that the way of water flowing into the building from openings must be considered precisely to calculate buoyancy. However, since the description hasn't been clarified yet, we decided to adopt the following method to consider buoyancy as an assumption for safe side.

a) In designing the superstructure for collapse prevention, the buoyancy of when calculating the horizontal force on each story, should include enough water flow into the building from the openings of each floor in proportion to the inundation depth. The buoyancy can be calculated as a sum of the buoyancy for the volume of the building elements under the inundation depth which works on the structure, and the buoyancy of dead air space under the floor slab. Since the axial force will be minimum when there is water in the building, the flexural strength of columns in RC structure and the lateral capacity on each story will be calculated as smaller. Therefore it can be said it's safer. Also if it is possible to calculate the lateral capacity of each story as safer, the buoyancy of the status can be used.

b) When designing a foundation for overturning and/or sliding prevention, the buoyancy in the calculation of the axial force on the piles or friction on the foundation should be calculated as taking the buoyancy for the amount of cubic content of the building for working on the bottom of the foundation. It was confirmed that the tsunami with a depth which was as deep as the maximum inundation depth impacted, at one push, into the Sendai Plain at this time. In this case, though the tsunami water went around buildings, not much water could flow into buildings. Therefore it can be said that buoyancy easily worked on the buildings. Also, since water flowing into buildings with fewer openings is always slow, it can not be said that water flows in according to the inundation depth. Hence, as a measure for safety by considering these uncertain elements, it should be a principle to consider the buoyancy for the amount of the cubic content of the building at this moment.

2.6 How to Deal with Debris Impact
There are various kinds of drifting objects in a tsunami disaster, such as driftwood, automobiles, containers, vessels, pieces of destroyed buildings. Several different methods to calculate the impact force of collision between a building and a drifting object are proposed. However, the calculated values of each method are very different from each other, and the kinds of objects were limited. Therefore, a united evaluation method, which can meet various situations, hasn't been established yet. Also, by calculating with the

	The inundation depth and the number of stories of a building		
	5m (4F)	10m (5F)	15m (7F)
a=3.0 Short direction (span=12m) Long direction (C_B=0.3)	◎ C_B=0.97 Length 40m	△ C_B=2.83 Length 36m (C_B=1.0)	△ C_B=4.56 Length 54m (C_B=1.0)
a=2.0 Short direction (span=12m) Long direction (C_B=0.3)	◎ C_B=0.38 Length 15m	○ C_B=1.44 Length 60m	△ C_B=2.42 Length 54m (C_B=0.55)
a=1.5 Short direction (span=12m) Long direction (C_B=0.3)	◎ C_B=0.30 Length 9m	◎ C_B=0.78 Length 33m	○ C_B=1.36 Length 54m

◎ means the level which can corresponded to the current earthquake resistance design. ○ means the level which requires some means to increase the strength, and △ means the level which requires special means to increase the strength of the superstructure, the piles, and the foundation decisively .

Figure 4. Base shear coefficient C_B in the short direction and length in the long direction which are required of the building for each inundation depth

methods which have been proposed thus far, in some cases when drifting wood or a container has hit, even a RC column could be failed [7].

Therefore it is decided to confirm that the axial force supporting capacity will not be lost by transferring it to next column or wall via beams if a column was failed by debris impact. Generally, it is not necessary to consider the situation in which two or more columns are failed at once. But, in case the drifting object must be huge, such as a vessel, we have to consider a method to confirm the building will not collapse if the exterior columns are destroyed. Also, to build protective equipment or facilities from debris impact could be a measure from the planning side.

3. CONDITIONS WHICH ARE REQUIRED FOR TSUNAMI EVACUATION BUILDINGS

The proposed structural design method for tsunami evacuation buildings was examined how much strength and size are required against a large inundation depth, like the one we experienced in the Great East Japan earthquake. In this examination, an RC apartment house with multistory walls in short direction with span of 12m and frame structure for the long direction with the demand base shear coefficient of 0.3 was taken, which meets code requirements for seismic safety. The water depth coefficient a=1.5, 2.0, and 3.0, the inundation depth = 5m, 10m, and 15m are assumed. Then the base shear coefficient (C_B) for the short direction to meet the conditions of this method was calculated, and also the length of the long direction for each combination of the water depth coefficient "a" and the inundation depth was calculated. The height of each story of the building is 3.5m, and the ratio of openings is 0.3 were assumed. According to the result, when the inundation depth was 5m, even by setting a=3.0, the C_B for the short direction is 0.97.

It is acceptable to calculate with the root 1 in seismic calculation, which is used for structures with many walls and requires $C_B \geq 1.0$. When the inundation depth was 10m, the short direction in case of a=1.5 can be correlated by the calculation by the root 1. But if it is a=2.0, the C_B of the short direction will be 1.44 and the required length of the long direction will be 60m. Therefore, the strength for both directions requires it to be much greater than the value in the usual earthquake-resistance design. Also, the short direction in case of a=3.0 requires C_B=2.83. So, some special means to make the strength greater will be needed not only for the superstructure but also for the piles and the foundation. If the inundation depth hit 15m, even when a=1.5 much greater strength than the usual cases will be required. If a=2.0 under the same condition, it is clear that some special means to increase the strength of the superstructure, the piles, and the foundation significantly, have to be taken. The short direction in case of a=3.0 requires C_B=4.56, which is extremely great in strength.

The above was an examination of collapse, but since the structural design method of a tsunami evacuation building requires so-called "secondary designing," which corresponds to the ultimate capacity design even on the piles and the foundation, the piles have to have much greater lateral capacity and tensile resistance, compared with one designed by the allowable stress calculation.

Figure 4. summarized what was mentioned in this chapter by ◎, ○, and △. ◎ means the level which can corresponded to the current earthquake resistance design. ○ means the level which requires some means to increase the strength, and △ means the level which requires special means to increase the strength of the superstructure, the piles, and the foundation decisively.

4. CONCLUSION

In this paper, the structural design method in "The Guidelines concerning the tsunami evacuation buildings etc.," which was issued in 2005 was reviewed. The results are reflected in "The interim guideline for the structural conditions of tsunami evacuation buildings etc." and Notification "Technical advice how to design a building which has a safe structural proof stress against tsunami." These are based on the examination in the fiscal year of 2011. However, concerning the structural design method of tsunami evacuation buildings, there are still many tasks to clarify, such as the tsunami wave pressure calculation by considering the tsunami fluid velocity, the condition of the calculation, how to treat the openings, measures for treating drifting objects, measures for scouring, and knowledge for how to treat piloti, and so on. We are going to continue our technical research to establish the rational method to calculate these phenomena properly.

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APPENDIX

TASK COMMITTEE REPORTS

Report of Task Committee A
STRONG MOTIONS AND EFFECTS

Date: 20 February 2013

Place: National Institute of Standards and Technology, Gaithersburg, MD, USA

Attendees:

U.S. Side --	Mehmet Celebi (Co-Chair)	USGS
	Asok Ghosh	VA
	Fred Lau	VA
	Steven McCabe	NIST
	Joy Pauschke	NSF
Japan Side --	Izuru Okawa (Co-Chair)	BRI
	Masanori Iiba (Co-Chair)	BRI

1. Objective and Scope of Work

The main objectives of the task committee are:

(1) To promote sharing of strong motion earthquake data among researchers and practicing engineers, and enhance the availability of technology for evaluating the destructive effects of earthquake motion.

(2) To promote, and when feasible, conduct collaborative research with other task committees of the Wind and Seismic Effects Panel on the dynamic behavior of structures.

(3) To promote and coordinate research on ground motion characterization, ground motion prediction and processing, and site-characterizations as applied to structural design considerations in building codes and other standards.

The scope of work includes:

(1) Exchange strong motion data and associated meta-data regularly and identify significant issues.

(2) Exchange information on technological developments, state-of-the-art and practice related to strong motion recording, archiving and processing, design ground motion estimation, hazard mapping, selection and modification of recordings for dynamic structural analysis, soil-structure interaction, soil behavior, and stability during earthquakes.

(3) Coordinate, and when feasible, plan and conduct programs of cooperative research and/or workshops in coordination with the proposed or ongoing programs. Disseminate results of workshops.

2. Accomplishments

(1) Information and data on the 2011 Great East Japan Earthquake were exchanged.

(2) Joint papers were published:

a) Okawa, I., Kashima, T., Koyama, S., Iiba, M. and Çelebi, M. 2012, Summary of recorded building responses during the 2011 Off the Pacific Coast of Tohoku earthquake with some implications to design motions, Proc. of International Symposium on Engineering Lessons Learned from the Giant Earthquake, March 2012

b) Çelebi, M., Okawa, I., Kashima, T., Koyama, S. and IIba, M., 2012, Response of a tall building far from the epicenter of the March 11, 2011 M=9.0 Great East Japan earthquake and its aftershocks, Paper 0291, Proc. of 15th World Conference on Earthquake Engineering.

c) Çelebi, M., Okawa, I., Kashima, T., Koyama, S. and IIba, M., 2012, Response of a tall building far from the epicenter of the March 11, 2011 M=9.0 Great East Japan earthquake and its aftershocks, Journal of Design of Tall Buildings and Special Structures, Published online in Wiley Online Library (wileyonlinelibrary.com/journal/tal), DOI: 10.1002/tal.1047

d) Çelebi, M., Okawa, I. and Kashima, T., 2012, March 11, 2011 M=9.0 Great East Japan Earthquake and Aftershocks: The story of a 9-story building retrofitted, damaged during, and repaired after the mainshock, Paper 0292, Proc. of 15th World Conference on Earthquake Engineering.

e) Çelebi, M. and Okawa, I., Drift issues of tall buildings during the March 11, 2011 M9.0 Tohoku earthquake, Japan - Implications, Paper presented during 44th UJNR Meeting (also to be submitted to journal).

3. Future Plans

(1) Joint studies on recorded motions obtained during the recent damaging earthquakes, including the 2011 Great East Japan Earthquake from buildings and other structures. The studies include/require:
   a) Exchange of data, meta-data related to the buildings and other structures from which data have been recorded.
   b) Understanding the effect of long-period ground motions to tall buildings and long-period structures.
   c) Determine the variation of structural characteristics of damaged buildings during the events.
   d) Study data for correlation with damage detection methodologies.
   e) Developing better instrumentation methods to obtain improved data during future events.
   f) Exchange of data on the near source ground motions to study impacts on design considerations.

(2) As a result of numerous recorded free-field data during the recent events:
   a) Study how they may affect design response spectra in Japan and the USA.
   b) Study site response issues, including topographical effect in particular, testing the transfer function procedures (e.g. Nakamura method)

(3) Exchange of information on the seismic hazard mapping for improving structural design

(4) Other activities as appropriate and events dictate

(5) Specifically during the next 2 years:
   a) Exchange further information and data on the 2011 Great East Japan Earthquake and other recent damaging earthquakes.
   b) Joint studies on building response and free field records.
   c) Exchange information on hazard mapping for building design.
   d) Look into reviving US-Japan UJNR SSI workshop within the next 2 years.
   e) Scaling for ground motions.

Report of Task Committee B
BUILDINGS

Date: 20 February 2013

Place: National Institute of Standards and Technology, Gaithersburg, MD, USA

Attendees:	U.S. Side --	Steven McCabe (Chair)	NIST
		Krishna Banga	VA
		Mehmet Celebi	USGS
		Fred Lau	VA
		H. S. Lew	NIST
		Joy Pauschke	NSF
		Kevin Wong	NIST
	Japan Side --	Masanori Iiba (Acting Chair)	BRI
		Izuru Okawa	BRI

1. Objective and Scope of Work

(1) Objective:
The objective of the Task Committee is to improve the seismic performance of buildings in the U.S. and Japan, thus reducing future earthquake damage to buildings. This Task Committee accomplishes this objective by promoting sharing technical information, performing appropriate cooperative research, exchanging personnel to address common issues, and working together to translate research results into the seismic provisions of codes and standards in the U.S. and Japan.

(2) Scope of Work:

a) Conduct joint workshops and meetings to identify new technical information and possible research cooperation/collaboration for the development of improved codes and standards.
b) Encourage the development, enhancement and application of new technologies and design methods to improve safety, sustainability and productivity of buildings and to improve the resilience of buildings and infrastructure.
c) Coordinate development of databases, test procedures, and guidelines for interpretation of test results and their applications.
d) Coordinate joint research including the utilization of experimental facilities.
e) Enhance the exchange of information and personnel.

2. Accomplishments

(1) The Task Committee conducted a U.S.-Japan joint reconnaissance on damaged buildings due to shaking of the 2011 Great East Japan Earthquake from August 31 through September 1, 2011 to investigate damage to retrofitted buildings, seismically isolated buildings, and buildings designed based on the current code or the previous code in Japan. A workshop on motion induced building damage was also conducted in Tohoku University with Prof. M. Maeda.

(2) Based on the August 2011 meeting, the US and Japan Buildings Task Committee groups began programs to study the behavior of reinforced concrete structural shear walls. This was identified as a priority for future collaboration.

3. Future Plans

(1) Create joint research between the US and Japan to develop and improve numerical models of structural elements and systems and to exchange experimental and field data. The following topics

have been identified as areas of future research collaboration on building structures:

a) Performance of RC structural shear walls; this is a priority.

b) Study of earthquake duration effects on structural performance; the relationship between the number of strong cycles of response and level of damage. This subject is an important part of Item a).

c) Resonant response of high-rise buildings and seismically isolated buildings to long-period earthquake ground motion. This will be a Task Committee A & B joint activity.

d) Examine the performance of seismically isolated buildings; summarize experience in US and Japan and modeling issues associated with this system.

(2) Exchange technical information on the following topics.

a) The strong motion data recorded in the buildings and its drawings

b) Structural performance data obtained by the tests conducted.

c) Provide links on NIST data repository when available.

(3) Future workshops, "U.S.-Japan Workshop on the Performance of RC Structural Shear Walls", will be developed to share technical information about the US and Japan research in this area. This information can be reflected within the technical codes and standards of the respective countries. Participation by Chilean engineers and researchers is also to be considered.

Report of Task Committee C
DAMS

Date: 20 February 2013

Place: National Institute of Standards and Technology, Gaithersburg, MD, USA

Attendees:	U.S. Side --	Enrique Matheu (Co-Chair)	DHS
		Bruce McCracken	USACE
	Japan Side --	Takashi Sasaki (Chair)	PWRI

1. Objective and Scope of Work

To promote better understanding of the response of dams to dynamic loads, the T/C will identify, coordinate, and support initiatives by government agencies, private sector, universities, research centers, and professional organizations to advance the safety and resilience of these critical structures, improve their performance under dynamic loading, promote effective remediation measures, and support emergency preparedness efforts.

The scope of work includes:

(1) Identify, review, and assess methods for dynamic analysis and performance evaluation of dams and related critical infrastructure (such as hydropower generation facilities, navigation structures, and flood risk reduction systems).
  - a) Assessment of models and numerical procedures used for non-linear response analysis of dams and related critical infrastructure.
  - b) Definition of input ground motions for non-linear seismic analysis.
  - c) Assessment of performance-based design and analysis approaches.
  - d) Development of effective counter-measures and retrofit alternatives to improve the performance under extreme dynamic loads.

(2) Identify, review, and assess physical modeling efforts supporting dynamic analysis and performance evaluation of dams and related critical infrastructure.
  - a) Determination of strength and deformation characteristics of concrete, soil, and rock materials under dynamic conditions.
  - b) Experimental evaluation of non-linear performance (e.g., shake table testing, centrifuge testing, etc.).

(3) Evaluate observed performance during earthquakes.
  - a) Development, review, and calibration empirical techniques for simplified assessment.
  - b) Review observed failure and damage mechanisms to improve the development of advanced numerical models.
  - c) Application of the analysis of the observed dynamic behavior to the improvement of design and evaluation criteria.

(4) Identify, review, and assess approaches to enhance emergency preparedness and mitigate potential consequences associated with incidents or events affecting dams and related critical infrastructure.
  - a) Evaluation of models and numerical procedures used for flood inundation modeling.
  - b) Evaluation of models and numerical procedures used for consequence estimation (human impacts and economic impacts).
  - c) Assessment of emergency preparedness approaches, including emergency action planning and exercises.

(5) Collaborate with universities, research centers, and professional organizations to promote information sharing across the dam engineering community.

2. Accomplishments

(1) Technical exchange and collaborative research on *Nonlinear Response Analysis and Discrete Element Method Analyses of Concrete Dams* has been conducted between the U.S. (U.S. Army Engineer Research and Development Center) and Japan (Public Works Research Institute). Shaking table experiments for crack-segmented concrete specimens considering the uplift pressure in a crack were successfully conducted at PWRI in 2009 and 2010. The U.S. Bureau of Reclamation is continuing efforts in nonlinear response analyses, and the joint comparison and evaluation of test and analysis results will be extremely beneficial to advance the state of the art in constitutive modeling of mass concrete structures.

(2) Technical exchange and collaborative research on *Experimental Characterization of Nonlinear Tensile Behavior of Mass Concrete* has been conducted between U.S. (U.S. Army Engineer Research and Development Center and U.S. Bureau of Reclamation) and Japan (Public Works Research Institute).

(3) The Task Committee extended an invitation to professional organizations, such as the U.S. Society on Dams and Japan Commission on Large Dams, to actively participate as members of the Task Committee by designating the corresponding representatives. The Task Committee has incorporated several new members from these professional organizations.

(4) The Task Committee conducted a U.S.-Japan joint reconnaissance on four dams damaged due to the 2011 Great East Japan Earthquake from August 31 through September 2, 2011.

3. Future Plans

(1) The Task Committee will continue current efforts focused on the development of improved mechanisms to facilitate the continuous exchange of results of research activities and general technical information related to the dynamic performance of dams and related critical infrastructure.

(2) The Task Committee will coordinate exchange visits of scientists and engineers from the U.S. and Japan. A series of case histories of mutual interest will be identified and prioritized and they will serve as the focus for this exchange program.

(3) The Task Committee will identify and promote collaborative opportunities on the following research areas:

a) ***Criteria for seismic analysis progression:***
The Task Committee will support the review and comparison of the state of practice in the U.S. and Japan regarding current recommendations for seismic analysis based on a systematic progression of analysis stages increasing in complexity.

b) ***Seismic evaluation of embankment dams:***
The Task Committee will support the review of criteria and guidelines for post-earthquake stability and deformation analysis of embankment dams.

c) ***Dam-foundation interaction:***
The Task Committee will support the development of improved numerical models for dam-foundation interaction.

d) ***Risk Assessment and Consequence Estimation:***
The Task Committee will support technical exchange and comparison studies related to risk assessment methodologies and consequence estimation models for dams and related critical infrastructure.

e) ***Flood Inundation Modeling:***
The Task Committee will support the review of the state of practice regarding numerical simulation techniques for flood inundation modeling.

(4) The Proceedings of the *4th U.S.-Japan Workshop on Advanced Research on Dams* will be published in 2013.

(5) The Task Committee will pursue collaborative efforts with the professional organizations, such as the U.S. Society on Dams and Japan Commission on Large Dams, and will seek to hold joint workshops, seminars, and other means of technical exchange in conjunction with their regularly scheduled

conferences and annual meetings.

(6) The Task Committee will approach the Committee on Earthquakes of the U.S. Society on Dams regarding the possibility of conducting a joint workshop in conjunction with the International Commission on Large Dams Annual Meeting to be held in Seattle, USA, during August 12-16, 2013. Workshop details will be determined through correspondence between the Chairs of these two committees.

(7) The Task Committee will explore future collaboration opportunities with the Committee on Computational Aspects of Analysis and Design of Dams of the International Commission on Large Dams. Task Committee members will participate at the *12th International Benchmark Workshop on Numerical Analysis of Dams*, to be held in Graz, Austria, during October 2-4, 2013, to discuss potential collaboration efforts.

Report of Task Committee D
WIND ENGINEERING

Date: 20 February 2013

Place: National Institute of Standards and Technology, Gaithersburg, MD, USA

Attendees:	U.S. Side --	Marc Levitan (Co-Chair)	NIST
		Partha Sarkar (Co-Chair) (via teleconference)	ISU
		Luca Caracoglia	NEU
		Kishor Mehta	NSF
	Japan Side --	Yasuo Okuda (Co-Chair)	NILIM
		Hitoshi Yamada (Co-Chair)	YNU

1. Objective and Scope of Work

To exchange technical information and to jointly plan, promote, and foster research and dissemination, in order to improve understanding of wind and its effects on structures, to establish more rational wind-resistant design procedures for structures, and to contribute collaboratively and synergistically to wind hazard mitigation. Specific objectives for the Task Committee include:

(1) Strategically and collaboratively, identify research needs in wind hazard mitigation in the areas of new impacts of wind events.
(2) Facilitate cooperation and collaborative research between U.S. and Japanese researchers in wind engineering.
(3) Identify and exchange successes in wind engineering and wind hazard mitigation.

The scope of the US-Japan collaboration includes:

(1) Characterization of strong wind, especially boundary layer extreme winds.
(2) The study of wind effects including wind loading on and wind-induced response of structures.
(3) Performance of experimental and analytical research to predict wind effects.
(4) Sharing damage surveys of wind hazard and storm surge and risk assessments in cooperation with Task Committee H.
(5) Development of new technologies for wind hazard mitigation.

2. Accomplishments

(1) Japan side Task Committee members shared information on tornado damage in Tsukuba City on May 6, 2012.
(2) US side Task Committee members shared information on tornado damage in Joplin Missouri on May 22, 2011.
(3) Follow-up study on the US-side, based on data from the "US-Japan Benchmark Study on Flutter Derivatives". (a) One journal paper published by researchers from Northeastern University (Seo and Caracoglia, Engineering Structures, 33 (2011) 2284–2296; (b) Short paper to be included in the proceedings of the 44th Panel Meeting from Northeastern University.
(4) Two workshops were held in November 2011 (Northeastern University, Boston, MA) and in March 2012 (Texas Tech University) in the United States on "Structural Dynamics and Monitoring of Bridges and Flexible Structures against Wind Hazards".
(5) A joint US-Japan manuscript was recently completed for submission to the ASCE Journal *Natural Hazards Review* titled "Wind-speed estimation and post-disaster recovery of building damage in the 2008 EF5-Tornado in Iowa, USA" by H. Kikitsu, BRI, Japan and P. P. Sarkar, ISU, USA (a Japanese version of this paper was published earlier: Kikitsu, H. and Sarkar, P. P. "Damage to Buildings by EF5 Tornado in Iowa, U.S. on May 25, 2008", Wind Engineers, JAWE, 33(4), 345-356). The above papers

are a result of a joint effort and damage survey of the Parkersburg, Iowa, EF5-tornado in 2008.

(6) A tornado simulator based on the design of Iowa State University Tornado Simulator was constructed in 2010-11 at the Building Research Institute, Tsukuba, Japan, under the supervision of Dr. H. Kikitsu in collaboration with Partha Sarkar of ISU, to conduct research on tornado-induced wind loads at BRI.

3. Future Plans

(1) The 6th US-Japan Workshop on Wind Engineering will be held in Yokohama in 2014. Discussion on the planning of this meeting:
  a) Propose to focus on few research items:
    • Tornadoes
    • Wind and wind-rain induced stay-cable vibration on long-span bridges
    • Performance of buildings under extreme wind loads
    • Wind energy systems
  b) Planning of the meeting is under way:
    • Two-day technical workshop and one day of technical tours
    • Optimal dates: third week of May in 2014 (possibly combining the UJNR Workshop with meeting of Japanese Association for Wind Engineering)

(2) Conduct collaborative research on the following topics. More concrete subjects were proposed at 5th US-Japan workshop in 2010.
  a) Wind effects on buildings and wind energy systems (land based and offshore)
    → Continuation of ongoing collaborative study of tornadic flow and effects on buildings structures
  b) Wind effects on bridges
    → Follow-up of benchmark study on flutter derivatives
  c) Evolving Technologies
    → Development of collaborative research on emerging innovative techniques for laboratory modeling and instrumentation

(3) Exchange technical information on the following topics.
  a) Wind characteristics and wind hazards
    → Conduct study on the urban flow using CFD simulation of flow over the cities
  b) Wind pressures, loadings and performance of buildings
    →Development of database of pressures on roofs and solar panels, resulting from comparative study carried out in Japan
  c) Wind-induced response of flexible, cable-suspended bridges and their components
  d) New prediction and mitigation techniques for wind effects
    →Use Cp pressure coefficient of hip roof and parapet
  e) Share the database of storm damage assessments with Task Committee H.
    → Establish wind induced damage database for buildings and infrastructure in Japan and US

(4) Engage in more regular interaction and communication among Task Committee members. Use email and exchange visits between full Panel meetings were suggested as a means of facilitating and coordinating collaborative activities.

(5) Exchange of graduate students for short-term (summer) projects at research institutions on both sides should be pursued.

4. Related Activities

(1) The AIJ committee has revised AIJ Recommendations for Loads on Buildings (2004 Edition). The new edition will be completed in 2014 concluding a guideline for estimation of wind loads by CFD and a manual for wind resistant design of components and claddings.

(2) The ASCE 7 Wind Load subcommittee began work in 2012 on a revision to the wind loading provisions of the ASCE 7 standard, to be published in 2016.

Report of Task Committee G
TRANSPORTATION SYSTEMS

Date: 20 February 2013

Place: National Institute of Standards and Technology, Gaithersburg, MD, USA

Attendees:	U.S. Side --	W. Phillip Yen (Chair)	FHWA
	Japan Side --	Tetsurou Kuwabara (Chair)	PWRI
		Jun-ichi Hoshikuma	PWRI

1. Objective and Scope of Work

The objectives of work include:

(1) To plan, promote and foster research on the behavior of transportation facilities when subjected to wind and seismic forces, and

(2) To disseminate research results and provide specifications and guidelines based on the Task Committee's findings.

The scope of work includes:

(1) To investigate existing and new bridges design, the behavior of whole bridge systems and/or single component of a bridge without limitation on their size and function.

(2) Personnel exchange for young engineers in sharing research activities and technical information.

2. Accomplishments

(1) The proceedings of the 27th US-Japan Bridge Engineering Workshop, which was held during 7-9 November 2011, in Tsukuba, Japan, were printed and distributed. The program and papers of the workshop were posted on the website of the Panel on Wind and Seismic Effects, UJNR at PWRI (http://www.pwri.go.jp/eng/ujnr/tc/g/tc_g.htm) and FHWA.

(2) The 28th US-Japan Bridge Engineering Workshop was held during 8-10 October 2012, in Portland, OR, U.S. The proceedings of this Workshop will be printed and distributed. The program and papers of the workshop will be posted on the website of the Panel on Wind and Seismic Effects, UJNR at PWRI and FHWA.

(3) The report of joint reconnaissance of highway bridge damage due to the 2011 Great East Japan Earthquake, which was performed during 3-6 June 2011, was posted on the website of the Task Committee G, Panel on Wind and Seismic Effects, UJNR, so that the selected significant pictures in high resolution format can be downloaded by worldwide researchers and engineers.

(4) Dr. Phillip Yen visited PWRI on 5 March 2012 to observe the experimental test of the tsunami effect on bridges and exchanged the technical information on the tsunami effect with PWRI.

(5) Prof. Kazuhiko Kawashima visited New Orleans, LA on 17 and 18 March 2012 and investigated the I-10 bridges damaged by 2005 Hurricane Katrina.

(6) Mr. Zenchary B. Haber, Ph. D. candidate, University of Nevada, Reno, visited PWRI on 31 August 2012 and exchanged the technical information on the seismic performance of the precast segmental bridge columns with PWRI.

(7) Both sides started the U.S.-Japan collaborative researches on study of tsunami effects on bridge performance.

3. Future Plans

(1) The 29th US-Japan Bridge Engineering Workshop will be held in October 2013, in Tsukuba, Japan. Specific program and itinerary will be proposed by the Japan-side Task Committee G with the

concurrence of the US-side Task Committee G.

(2) Following a devastating earthquake or hurricane (typhoon) in the US or Japan, the committee will form a joint reconnaissance team to investigate the performance of transportation systems.

(3) With increasing concerns over structural member fractures of older bridges in the US and Japan, the committee will conduct joint efforts to investigate detection methods, causes and repairs. The joint efforts should be initiated by the hosting side.

(4) Both sides agreed to conduct joint researches and share technical information on the following topics.

a) Strategy to determine design criteria, design loads, and load factors that consider ductility and redundancy for multiple hazards

b) Best and poor practices in bridge design and maintenance

c) Post earthquake response and repair

d) Study on policy making to set different performance levels of routes and allocate resources for seismic upgrading/retrofit, bridge inspection, and rehabilitation based on the assigned characteristics

e) Impact of seismic design of long duration earthquakes

f) Applications of high performance materials (Nano, SMA and UHPC) in seismic design and retrofitting

g) Study of tsunami effects on bridge performance in cooperation with Task Committee H

4. Related Activities

None.

Report of Task Committee H
STORM SURGE AND TSUNAMI

Date: 20 February 2013

Place: National Institute of Standards and Technology, Gaithersburg, MD, USA

Attendees:	U.S. Side --	Marc Levitan (Chair)	NIST
		Kishor Mehta	NSF
	Japan Side --	Takashi Tomita (Chair)	PARI
		Yasuo Okuda	NILIM

1. Objective and Scope of Work

The objectives of work include:
(1) To exchange scientific and technical information
(2) To jointly plan, promote and foster research and dissemination of knowledge
(3) To develop measures to prevent and mitigate damages from storm surges and tsunamis

The scope of work includes:
(1) Perform joint research on storm surge and tsunami occurrences, generation, propagation, and coastal effects. Develop database on storm surge, tsunami and wave measurements.
(2) Improve coordination of strategies and systems for observations of storm surges and tsunamis by field surveys, satellites, and in-situ measurements.
(3) Exchange results and status of storm surge and tsunami mitigation activities including analysis of the problem, planning, warning, and engineering approaches.
(4) Exchange information on development of technologies including numerical models to predict propagation processes, landfall locations, inundation and run-up heights, and wave characteristics, improved instrumentation, and use of satellite communication for detection and warning.
(5) Facilitate research result and technology development disseminations through exchange of literature, technical reports at joint meetings, special workshops, joint projects, and direct interaction among participants.
(6) Develop planning, design and construction guidelines in storm surge and tsunami flooding zones to serve as a model for international standards.
(7) Provide technical support to develop storm surge and tsunami mitigation programs worldwide.
(8) Encourage conduct of joint investigation following storm surge and tsunami events in cooperation with Task Committees D and G.

2. Accomplishments

(1) Collaboration with T/C G on Bridges on tsunami impact design is in progress.
(2) Japan side members are participating with US side T/C members on the committee developing tsunami design provisions for the ASCE 7 standard applicable to buildings and other structures.
(3) Panel members of both US and Japan participated in numerical simulations of tsunami propagation and structural damage of the 2011 Great East Japan Earthquake Tsunami.
(4) Panel members of both US and Japan exchanged information on tsunami loads on buildings.
(5) Panel members from the US side exchanged information on Hurricane Sandy storm surge flooding.

3. Future Plans

(1) Create joint research between the US and Japan to develop and improve numerical models of storm surge and tsunami dynamics and to exchange experimental and field data. The following topics have

been identified as areas of future research collaboration on storm surges and tsunamis:

a) field observation
b) characterization
c) physical experiment models
d) numerical simulation models
e) effects on coastal structures and damage estimations
f) design of protective structures for different levels
g) hazard maps development and warning system design
h) storm surge and tsunami information communication and warning systems development
i) risk assessment including hazard beyond designed levels

(2) Develop database for existing and planned experiments including description and parameters of experiments to maximize overall available experimental data for understanding of physical behavior, numerical model validation and structural design.

(3) Include the effects of global warming on atmospheric and oceanographic environmental conditions leading to changes in the probability of occurrence and intensity of typhoons, cyclones and hurricanes, and sea level rise. These changes in typhoon and sea level characteristics will directly influence the characteristics and induced damages of future storm surges and tsunamis.

(4) Collaborate with T/C D on Winds to develop storm surge research.

(5) Explore possibility of holding a UJNR Panel Meeting at a future natural hazard conference.

4. Related Activities

(1) Japan side T/C members have cooperative research activities with the Technical Committee on Estimation and Reduction Technologies on Multi-Hazards of Earthquake and Tsunami, Japan Society of Civil Engineers and the Working Group of Tsunami Loads, Architectural Institute of Japan.

(2) Japan side T/C members have cooperation with Japan local and central government on planning of recovery and reconstruction policies, and improvement of design codes of buildings, bridges and other structures.

(3) US side T/C members have several on-going research projects on tsunami and storm surge numerical modeling and experiments at the HWRL of OSU, and research at NIST on risk quantification for design of coastal structures exposed to combined hurricane wind and storm surge effects.

CHARTER

CHARTER OF THE UJNR PANEL ON WIND AND SEISMIC EFFECTS

OBJECTIVES

1) Encourage, develop, and implement the exchange of wind and seismic technology between appropriate United States and Japanese organizations to share scientific and technological knowledge.

2) Develop strong technical links of scientific and engineering researchers between the two countries and encourage exchanges of guest researchers.

3) Conduct joint research in areas of winds and seismic technology including exchange of available research equipment and facilities in both countries.
Publish findings from joint research efforts.

4) Conduct cooperative programs to improve engineering design and construction practices and other wind and earthquake hazard mitigation practices.
Publish results from cooperative programs.

CURRENT TOPICS AND SUBJECT AREAS OF INTEREST

1) Strong Motion Instrumentation Arrays and Data

2) Large Scale Testing Program

3) Repair and Retrofit of Existing Structures

4) Evaluation of Performance of Structures

5) Natural Hazard Assessment and Mitigation Through Land Use Programs

6) Disaster Prevention Methods for Lifeline Systems

7) Wind Characteristics and Structural Response

8) Soil Behavior and Stability During Earthquakes

9) Storm Surge and Tsunamis

10) Wind and Earthquake Engineering for Transportation Systems

COOPERATIVE ACTIVITIES

1) Conduct annual joint panel meetings alternating locations between the United States and Japan.

2) Publish proceedings of annual meetings and of task committee events.

3) Exchange data and information between both countries.

4) Exchange guest scientists and engineers.

5) Develop cooperative research programs on mitigating the effects of wind and seismic forces on structures. Concerning these programs, exchange available research equipment and facilities in both countries, if necessary.

6) Conduct task committee meetings and workshops in areas identified in “Current Topics and Subject Areas of Interest” to facilitate exchange of technical information.

7) Establish and maintain effective communications between scientists, engineers, and administrators of the two countries.

PANEL MEMBERSHIP

1) Members of the panel are personnel of government agencies designated by the agencies.

2) Other experts may be selected, as temporary members, from appropriate disciplines representing industry, academia, and research organizations.

CHARTER MODIFICATIONS

This Charter may be revised by the concurrence of the US and the Japanese sides.