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**PROCEEDINGS OF
THE 2ND U.S.-JAPAN WORKSHOP ON
DESIGN FOR WIND AND
WIND HAZARD MITIGATION**

May 24-26, 1999

**PUBLIC WORKS RESEARCH INSTITUTE
MINISTRY OF CONSTRUCTION**

1, Asahi, Tsukuba-shi, Ibaraki-ken, 305-0804 Japan

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DESIGN FOR WIND AND
WIND HAZARD MITIGATION**

Edited by:

Hiroshi Sato

Synopsis:

The proceedings document the results of the 2nd U.S.-Japan Workshop on Design for Wind and Wind Hazard Mitigation, which was held at the Public Works Research Institute, Tsukuba Science city, Japan, on May 24-26, 1999 under auspices of the Panel on Wind and Seismic Effects, UJNR. The workshop was organized by the Task Committee E "Design for Wind and Wind Hazard Mitigation" (U.S. side chairman: J. Golden, NOAA, Japan side co-chairmen: H. Okada, BRI and H. Sato, PWRI) of the Panel.

Key Words: Wind Characteristics, Wind flow, Wind pressure, Wind loading, Wind-induced response, Wind hazard, UJNR



Second Row: K. Ogi, T. Tamura, K. Kimura, L. Phan, P. Sarkar N. Jones, D. Reed, G. Chiu, T. Mabuchi, N. Furuya, N. Kinoshita, B. Bienkiewicz, Y. Tanaka, E. Ritchie

First Row: Y. Tamura, A. Kareem, M. Gaus, H. Okada, H. Sato, J. Golden, Y. Inoue, A. Chiu, M. Yagi, M. Powell

Table of Contents

1. Overview	1
2. Resolutions	5
3. Group Reports	9
4. Technical Presentations	17
Session1 Wind Characteristics	19
(1) The wind engineering of extreme windstorms: How well do we know the windspeed distributions? (J. Golden)	21
(2) 10km and under 1km scale wind speed fluctuation observed in typhoon area (Y. Tanaka).....	27
(3) Understanding wind speeds in hurricanes: Implications for meteorological and engineering applications (M. Powell, T. Reinhold and J. Golden)	31
(4) A study on spatial correlation of natural wind (N. Furuya and R. Toriumi)	35
(5) A modeling study of the development of mesoscale asymmetries in tropical cyclone structure during landfall (E. Ritchie and R. Elsberry).....	39
Session 2 Wind Flow, Pressure and Loading.....	43
(1) Comparison of wind pressure measurements on a tower-like structure between full-scale observation and wind tunnel test (H. Okada, K. yasui, K. Yoshikawa, M. Watakabe, M. Okabe and H. Kikitsu)	45
(2) Roof corner wind flow and pressures (P. Sarkar, D. Smith and K. Mehta).....	53
(3) Extreme load distributions on buildings with a rectangular plan (Y. Tamura, H. Kikuchi and K. Hibi)	57
(4) Performance-based objective design and natural disaster risk management (G. Chiu).....	63
(5) Feasibility of CFD technique for prediction of wind flows and wind loading (T. Tamura).....	67
(6) Towards refined modeling of wind effects through collaborative research (B. Bienkiewicz).....	75
Session 3 Wind-induced Response	79
(1) Long-span bridge response to wind: Recent developments (N. Jones, R. Scanlan and H. Katsuchi).....	81
(2) Wind-induced response characteristics of bridges in the wake of a hill (K. Kimura, U. Roy and Y. Fujino)	85

(3) Effect of turbulence on the space-time characteristics of the self-excited forces on a rectangular cross-section (F. Haan, A. Kareem, and A. Szewczyk)	89
(4) Cooperative study between Public Works Research Institute and Honshu- Shikoku Bridge Authority on wind resistant design of super long-span bridges (H. Sato and N. Furuya)	93
(5) Development of computational models for bridge deck motion (T. Mabuchi, H. Sato, D. Reed, G. Turkiyyah and H. Yeh).....	97
(6) Experimental study on reducing aerodynamic response of tall buildings with open passage design (H. Kikitsu).....	101
(7) Wind-induced dynamic behavior of a single-layer latticed dome (Y. Uematsu, A. Sasaki and M.Yamada).....	105
Session 4 Wind Hazards.....	109
(1) Damage to structures caused by amplified wind speeds in complex topographical terrain (Arthur Chiu).....	111
(2) Damages due to strong wind of typhoon 9807 in Kansai area (M. Matsumoto, H. Shirato and T. Yagi).....	115
(3) GIS-based wind vulnerability analysis of urban tree populations (M. Gaus and D. Liang).....	119
(4) The fujita tornado intensity scale: A reassessment from structural Engineering perspective (L. Phan and E. Simiu).....	123
Appendix A: Workshop and Study Tour Programs	131
Appendix B: List of Participants	137
Appendix C: Photos of Workshop and Study Tour.....	145

1. Overview

OVERVIEW

The Second US-Japan Workshop on Design for Wind and Wind Hazard Mitigation was held on 24-26 May 1999, at the Public Works Research Institute. The Workshop was organized by the Structure Division of PWRI under the auspices of Task Committee E (US-side chairman: Dr. J. H. Golden, National Oceanic and Atmospheric Administration, Japan-side Chairmen: Dr. H. Okada, Building Research Institute, and Dr. H. Sato, PWRI) of the Panel on Wind and Seismic Effects, UJNR. Twelve US-side Task Committee members and 15 Japan-side members participated in the Workshop.

After the opening remarks by Mr. Y. Inoue, Director-General of PWRI, 22 participants presented papers covering wind characteristics; wind flow, pressure and loading; wind-induced response; and wind hazards.

On the second day of the Workshop, the participants separated into three groups: wind characteristics and wind hazards; wind effects on buildings; and wind effects on bridges. Each group discussed and identified research topics of mutual interest. The results were presented and discussed on the third day of the Workshop, and resolutions were adopted.

In the resolutions, the following technical topics for cooperative research programs were recommended:

In the field of wind characteristics and wind hazards,

- a. Wind effects due to complex terrain
- b. Modeling and validation with full-scale data
- c. Joint field studies
- d. Cooperative quick-response post-storm damage assessments

In the field of wind effects on buildings,

- a. Development of realistic aerodynamic loads for performance-based structural design

In the field of wind effects on bridges,

- a. Improvement of prediction methods for wind-induced response of cable-suspended bridges with edge girders
- b. Prediction and mitigation of wind-induced vibration of stay cables

It was also recommended to conduct the Third Workshop in the autumn of 2001, possibly at the University of Washington in the US.

Before and after the Workshop, the participants visited the following technical sites: wind tunnels of Meteorological Research Institute, BRI and PWRI; and the Honshu-Shikoku bridges between Onomichi and Imabari.

2. Resolutions

**Resolutions of the Second U.S.-Japan Workshop of Task Committee E,
Design for Wind and Wind Hazard Mitigation,
of the Panel on Wind and Seismic Effects, UJNR
Tsukuba, Japan
May 24-26, 1999**

The following resolutions and recommendations are hereby adopted:

1. The Workshop provided valuable exchange of technical information which was beneficial to the wind engineering communities of both countries. Twenty-two participants representing government agencies and academia presented papers covering: wind characteristics; wind flow, pressure and loading; wind-induced response; and, wind hazards. In view of the importance of cooperative programs on the subject of wind effects on structures, the continuing technical cooperation and interaction are essential.

2. The Workshop recognizes the importance of exchanging technical information, meteorological data and structural response data from strong winds, and of using available unique research facilities in both countries. It encourages cooperative efforts in mutually agreed-upon areas of wind engineering research to further knowledge of strong wind and the effects of wind.

3. The Workshop recommends the following technical topics for cooperative research programs:

3.1 Wind characteristics and wind hazards

- a. Wind effects due to complex terrain
- b. Modeling and validation with full-scale data
- c. Joint field studies
- d. Cooperative quick-response post-storm damage assessments

3.2 Wind effects on buildings

- a. Development of realistic aerodynamic loads for performance-based structural design
 - a-1 Estimation of cross-wind and torsional effects
 - a-2 Damping estimation of structures---data collection, analysis and cataloging
 - a-3 Comparison of gust factors and wind pressure coefficients of various international building codes
 - a-4 Full-scale measurements of wind-induced loads on low-rise construction
 - a-5 Examination of turbulence effects on wind loading and structural response by Computational Fluid Dynamics (CFD) and experimental methods
 - a-6 Quantifying "damage" for performance-based design

3.3 Wind effects on bridges

- a. Improvement of prediction methods for wind-induced response of

cable-suspended bridges with edge girders

Field measurement

Section model test/Aerodynamic derivative

Elastic model test

Analysis

CFD

Possible international workshop on aerodynamic derivatives

b. Prediction and mitigation of wind-induced vibration of stay cables

Field measurement

Section model test/Aerodynamic derivative

Elastic model test

Analysis

Mitigation

4. It is anticipated that the interaction among the delegates from the United States and Japan during the discussions and the technical study tours will lead to an enhanced exchange of data and innovative ideas, and that this interaction will also lead to the establishment of cooperative research efforts, both theoretical and experimental, on wind engineering. Because of the importance, benefits and potential of these Workshops, it is recommended to conduct the 3rd Workshop in the autumn of 2001, possibly at the University of Washington in the U.S. A study-tour will be planned to visit research laboratories and sites at which wind-engineering related projects are performed.

Adopted by the participants on May 26, 1999.

3. Group Reports

Group A: Wind Characteristics and Wind Hazards

Group B: Wind effects on Buildings

Group C: Wind Effects on Bridges

Report of Group A: Wind Characteristics and Wind Hazards

Group Members:

Arthur Chiu, UHM
Michael Gaus, SUNY, Buffalo
Joseph Golden, NOAA (Chair)
Mark Powell, NOAA
Elizabeth Ritchie, NPS
Ken-ichi Ogi, PWRI
Yukio Tamura, TIP
Masamitsu Yagi, MRI

Exchange of Information:

M. Gaus: What new efforts either side hasn't heard about?

J. Golden: Look at last workshop report and explore status of ongoing projects with potential for joint work.

Y. Tamura: Field program. Central Research Institute of the Electric Power Industry collects data sets on typhoons (1992-1999) in Shikoku. This site has sampled several typhoons with winds of 40-50 m/s. These data may be made available, via Dr. Y. Tamura.

MRI has 213 m tower. They need to check to see if similar wind maxima are present in their data. The workshop identified Doppler studies of coherent linear structures being conducted by US (OU, NOAA) and Japan sides (Tanaka).

Also K. Knupp needs to know about Doppler sodar. US side will be made aware of D. Sodar. Y. Tamura will provide data and references. Also Dr. Yagi mentioned that MRI has a profiler.

Complex Terrain:

Wind effects of complex terrain were discussed. One aspect of discussion was the need to educate the public and building officials about the risk of building in areas susceptible to extreme winds induced by terrain. Maps depicting "hot spots" of terrain speed up (high wind zone map, like a Tsunami inundation map) might be one way to address this issue. T. Tamura mentioned terrain CFD in his paper. Full scale data are required to validate CFD and wind tunnel studies (all longer term). Y. Tamura says complex terrain effects are similar to shape factors of a building and very difficult to generalize. However, we can initiate a process. Two Architecture Institute of Japan reports, from the wind tunnel and numerical modeling studies of simple topographic shape effects on winds, are in final draft. These reports will provide revised wind load recommendations for 2003. Published data will be available.

Modeling:

Several mesoscale numerical models are being used for tropical cyclone related studies that may be adapted to examine complex terrain issues by incorporating high resolution terrain databases. Dr. Yagi, JMA has many data available for validating E. Ritchie's mesoscale modeling work. In addition, data may be available from the Kyushu Mesoscale network (138 stations wind only). Y. Tamura knows who is managing the data.

Wind Hazards:

In Japan, structural response is being measured on several high rises but not wind pressures. Using peak gusts to predict damage doesn't seem to work very well anymore as structures become more complex. Y. Tamura says peak gusts can establish losses statistically on a prefecture scale (insurance company); but for individual buildings, detailed studies are required.

Field Programs:

MRI is planning to test Aerosonde in Okinawa once the Iridium communications system comes online. Maybe two aerosondes for typhoon and winter storms. Adeos II will be launched next year with NASA Scatterometer.

One Japanese group is wind tunnel testing low rise structures.

The NOAA Hurricane Research Division (HRD) field program has made some of their datasets available via the web (www.aoml.noaa.gov/hrd then go to storm atlas). Lead project scientist reports and 1-min data are available now and a longer term dataset should also be available of GPS sondes and processed flight level data binned by radial passes through the tropical cyclone center.

Japan Association for Wind Engineering (JAWE) has a quick-response network set up (set up in March, guideline report in Japanese). Ministry of Education could supply additional funds on a case by case basis. Sometimes MRI and JMA participate in damage surveys. Possibility of US wind engineers joining JAWE team and that of Japanese engineers joining the US team should be considered. Initiate cooperative quick-response teams for damage assessments and perishable data. Possibly set up a protocol for doing joint surveys to facilitate this effort. The disaster investigation committee in JAWE is chaired by Prof. T. Ohkuma, and vice-chaired by Y. Tamura. Incidentally, Dr. Katsura from Disaster Prevention Research Institute of Kyoto University is going to visit Oklahoma damage caused by the recent tornado.

ASCE and NWS have reports on how to do storm damage surveys. Most NOAA NWS assessments are to evaluate products and services.

Report of Group B: Wind effects on Buildings

Group Members:

Gregory Chiu, CUHK
Ahsan Kareem, UND
Long Phan, NIST
Hiromitsu Kikitsu, BRI
Hisashi Okada, BRI (Chair)
Tetsuro Tamura, TIT
Yasushi Uematsu, TU

Objective:

Development of realistic aerodynamic loads for performance-based structural design

Overall approach:

Aerodynamic Loads

Hazard Analysis
Load Evaluation

Tools

Numerical Simulation
Wind Tunnel Testing
Full-Scale Measurement

Tasks:

Identification of an institution(s) with each subtopic does not necessarily commit that institution to that topic, nor preclude the participation of other interested parties. All topics have at least one institution with primary or shared responsibility for that topic. Actual participation will be contingent on availability of research funds to support the research efforts on both the US and Japan sides.

Estimation of cross-wind and torsional effects (UND)

Damping estimation of structures (JHU, TIP, UND)

Data collection, analysis and cataloging

Comparison of gust factors and wind pressure coefficients in various international building codes (BRI, NIST, UND)

Full-scale measurements of wind-induced loads on low-rise construction (TTU, JHU, NIST)

Examination of turbulence effects on wind loading and structural response by CFD and

experimental methods (TIT, BRI, CSU, NIST, UND)
Quantifying "damage" for performance-based design (CUHK, BRI, NIST)

Abbreviations:

BRI	Building Research Institute
CSU	Colorado State University
CUHK	City University of Hong Kong
JHU	Johns Hopkins University
NIST	National Institute of Standards and Technologies
TIP	Tokyo Institute of Polytechnics
TIT	Tokyo Institute of Technology
TTU	Texas Tech University
UND	University of Notre Dame

Report of Group C: Wind Effects on Bridges

Group Members:

Bogusz Bienkiewicz, CSU
Nick Jones, JHU
Dorothy Reed, UW
Partha Sarkar, TTU
Kichiro Kimura, UT
Toshiaki Mabuchi, PWRI
Hiroshi Sato, PWRI (Chair)
Hiromichi Shirato, KU

The group identified two primary general research topics of mutual interest. Each of these topics is multi-component, with participation from a number of institutions envisioned.

Identification of an institution(s) with each subtopic does not necessarily commit that institution to that topic, nor preclude the participation of other interested parties. All topics have at least one institution with primary or shared responsibility for that topic. Actual participation will be contingent on availability of research funds to support the research efforts on both the US and Japan sides.

Both topics represent research thrusts in which current or proposed efforts are underway by various participants. It is hoped that success in funding of individual participants can thereby be used to assist in the leveraging of funding for other groups.

Improvement of prediction methods for wind-induced response of cable-suspended bridges with edge girders:

Group members identified a number of issues that needed attention in the prediction of long-span bridge response to wind including: full-scale validation of section-model-based analytical approaches and full-bridge models; the development of CFD techniques to predict aerodynamic and aeroelastic loads on cross sections; and the investigation of the potential use of more economical cross sections (traditional H section) in future structures. Using measurements on full scale structures (e.g., the Houston Ship Channel Bridge and the Tacoma Narrows Bridge) and other documentation as a base, a number of specific efforts were envisioned including:

- Field measurement (JHU,UW)
- Section model tests/aerodynamic derivatives (TTU, PWRI, UND)
- Elastic model tests (CSU)
- Analysis (JHU, UT, UND)

CFD (KU, UW, PWRI, UND)

Possible international workshop on aerodynamic derivatives (JHU+others)

Prediction and mitigation of wind-induced vibration of stay cables:

Large-amplitude vibration of the stays of cable-stayed bridges has occurred in a number of bridges around the world and is currently a focus of intense research efforts as both countries struggle to fully understand the mechanisms inducing the motion and means by which they can be reduced. Building on current and proposed efforts in both countries, the group suggested the following subtopics as possible collaborative foci:

Field measurement (JHU, TTU)

Section model tests/aerodynamic derivatives (KU, TTU, UND)

Elastic model tests (KU, TTU, CSU)

Analysis (JHU, KU, UND)

Mitigation (UW, PWRI, KU, UT, TTU, CSU, JHU)

Abbreviations:

CSU	Colorado State University
JHU	Johns Hopkins University
KU	Kyoto University
PWRI	Public Works Research Institute
TTU	Texas Tech University
UND	University of Notre Dame
UT	University of Tokyo
UW	University of Washington

4. Technical Presentations

Session 1 Wind Characteristics

(Chaired by M. Yagi)

- (1) The wind engineering of extreme windstorms: How well do we know the windspeed distributions?

(J. Golden)

- (2) 10km and under 1km scale wind speed fluctuation observed in typhoon area.

(Y. Tanaka)

- (3) Understanding wind speeds in hurricanes: Implications for meteorological and engineering applications.

(M. Powell, T. Reinhold and J. Golden)

- (4) A study on spatial correlation of natural wind.

(N. Furuya and R. Toriumi)

- (5) A modeling study of the development of mesoscale asymmetries in tropical cyclone structure during landfall.

(E. Ritchie and R. Elsberry)

The Wind Engineering of Extreme Windstorms: How Well Do We Know the Windspeed Distributions?

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ABSTRACT. New opportunities for joint US/Japanese cooperative research to improve the forecasting and surface measurement of extreme winds in landfalling tropical cyclones and other severe storms are outlined; these include the application of portable Doppler radars and rugged anemometers to map details of hurricane winds in the boundary layer and for critical time-series data to validate gust-factor models. Such field studies should guide the application of high-resolution numerical simulation models to the extreme windflow problem over complex terrain, and to wind-tunnel testing for wind engineering mitigation measures. Some lessons learned from quick-response surveys of damage after recent U.S. hurricanes are presented, including the importance of topographic effects on wind damage to residential construction. Some wind engineering methods applied to providing occupant protection and mitigation against windstorm damages are described.

INTRODUCTION

The primary purpose of this brief overview paper is to summarize recent advances in the U.S. for better observing and monitoring of severe storms, especially those accompanied by extreme winds. This comes at a time in which losses from natural disasters in the U. S. are approaching *one billion dollars every week*, moreover, more yearly fatalities in the U.S. are due to floods and lightning than to windstorms (tornadoes and hurricanes). It might be argued that this situation has partly resulted from the significant improvement in the accuracy and lead-times for tornado and hurricane warnings associated with the National Weather Service's Modernization. The U.S. Property Claims Services' insurance-loss claims from all windstorms have risen from under \$1 billion in the late 1980's to over \$5 billion in 1992 (much of that due to Hurricane ANDREW, which produced \$25 billion in total damages), and nearly \$5 billion in 1994 and 1995 as well.

SOME RECENT NOTABLE EXTREME WIND EVENTS

Up to the early part of this decade, the U.S. National Research Council's Committee on Natural Disasters (CND) sponsored quick-response scientific study teams that were dispatched to the sites of natural disasters. One of the primary objectives of these teams was to gather perishable scientific data and write a report documenting the impacts of the natural disaster. The present author is a past Chairman of the now defunct CND, and participated in a number of their disaster investigations, including Hurricane ALICIA (1983) along the upper Texas coast and Hurricane HUGO (1989) in the eastern Caribbean. The complete NRC/CND team reports for these hurricanes are available from the National Academy of Sciences, and the meteorology of Hurricane HUGO was described in detail by Golden (1990) at the 22nd Joint UJNR Panel Meetings.

The first NOAA reconnaissance aircraft was able to reach HUGO at mid-day on September 15, 1989 when the storm was east of the island of Barbados. It made the first penetration of HUGO's eye at an altitude of 1500 ft. and measured a central pressure in the eye of 918 mb, and one-minute flight-level windspeeds of 82 m/s. These data indicated that HUGO had intensified to a rare Category 5 status on the Saffir-Simpson scale. Some of the problems in relating aircraft reconnaissance winds, usually measured at 10,000 ft. to surface winds measured by anemometers in hurricanes have been addressed by Powell (1980), Black et al (1988) and Powell and Black (1990). Hurricane HUGO weakened slightly before striking Guadeloupe and then St. Croix and St. Thomas as a Category 4 hurricane. The team examined the extensive damage on the latter two islands, and found much of it heavily influenced by the island topography. We also found evidence that much of the most severe wind damage was produced by "downburst" phenomena induced by major hurricane rainbands, and documented by Fujita (1985) in severe midlatitude thunderstorms. The team found an appalling lack of recorded meteorological surface data, especially winds, over Puerto Rico, St. Croix and St. Thomas. As noted in earlier NAS/NRC hurricane study-team reports (e.g., Savage et al, 1984 on Hurricane ALICIA), many anemometers were in place, but most were either damaged or destroyed by HUGO's winds, or lacked recording equipment and/or back-up power. Only two verifiable wind records were obtained, and both were from the northeast coast of Puerto Rico (see Marshall, 1990, for wind plots). We had to reconstruct probable one-minute sustained windspeeds and peak gusts for the other nearby islands, using our aerial damage survey and reduction of reconnaissance aircraft-measured winds at 10,000 ft down to the surface, using the research results of Powell and Black (1990). Overall, we believe that the highest gusts in Hurricane HUGO occurred in Culebra and St. Croix (estimated peak gusts to 67 and 70 m/sec, respectively).

Hurricane ANDREW hit South Florida some 13 years after HUGO devastated the northeast Caribbean and then South Carolina, and once again, there were very few surviving wind records at the surface. Damage in South Florida, especially in South Miami amounted to \$30 billion, making ANDREW the mostly costly natural disaster in U. S. history. Subsequent analyses pieced together the few anemometer records, weather radar data and damage surveys to estimate windspeeds at landfall (Wakimoto and Black, 1993, Rappaport, 1994 and Fujita, 1992). While there was some disagreement between the meteorologists and wind engineers, most agreed that the highest sustained one-minute windspeeds were about 65 m/s, with gusts to 80 m/s in a few very localized regions affected by the eyewall passage. *Some of the apparent disagreements in damage/wind interpretation from ANDREW was due to the fact that wind engineers and meteorologists did not survey the damage together, but at different times and locations after the event.*

Both HUGO and ANDREW experiences underscore the need for coordinated, quick-response teams of interdisciplinary scientists, especially wind engineers, to be dispatched to the impacted region *no later than 24-36 hours after the event*. This is because we have found that much of the damage after major windstorms is rapidly cleaned up, and therefore lost for windspeed assessments.

Another major wind event that produced much more widespread wind and other damage was the so-called SUPERSTORM of March 12-14, 1993. The author participated as the Team Chief Scientist in a NOAA Disaster Survey Team, and its report was published in May, 1994. There were 79 deaths, 600 injuries and over \$2 billion in damage caused by the SUPERSTORM. This giant storm originated over the Gulf of Mexico as a frontal wave with wind-gusts above hurricane force, and deepened to a record 960 mb low-pressure area over the Chesapeake Bay. At its peak, it produced wind gusts of 25 m/sec or higher over a large part of the Eastern U. S. and heavy snow as far as 600 miles from the snow center (over 50" in the mountains of North Carolina). The

single most deadly phenomenon associated with the SUPERSTORM was the coastal flooding and unprecedented 3.6 m storm surge that occurred in the pre-dawn hours on March 13, 1993. Eleven people were drowned by the surge and to our knowledge, *no winter storm has ever produced a deadly storm surge of such magnitude in the Gulf of Mexico. In fact, the SUPERSTORM surge killed more people from drowning than Hurricanes ANDREW and HUGO combined.* This unusual storm-surge event has stimulated new research efforts by Dr. W. Shaffer and his colleagues in NOAA to produce a storm-surge prediction model for extratropical cyclones. Other deaths and property damage resulted from moderate-to-strong tornadoes and extremely fast-moving severe thunderstorms over Florida. A total of 11 tornadoes raked the north half of the Florida peninsula during the night on March 12- 13, killing 5 persons, injuring 88 others, and causing over \$66 million in damage. It is a remarkable aspect of this storm that nearly every county in the state of Florida received some form of wind damage, primarily from strong gradient winds and convective wind-gusts to 37 m/s.

NEW TECHNOLOGIES AND THEIR APPLICATION TO WIND DAMAGE MITIGATION

Two new technologies are available for application to the critical issue of improved ground-level wind measurements in severe storms: portable Doppler radars and ruggedized anemometers. A new 3-cm portable Doppler radar (called the "Doppler-on-wheels" or DOW) has been developed in a cooperative effort by the NOAA National Severe Storms Lab, the University of Oklahoma and NCAR. It has been used to intercept tornadoes and successfully acquire detailed radial velocity and reflectivity data at close range to a tornado during the 1994-95 VORTEX project (Straka et al, 1996). This same system was also successfully deployed at Wilmington, NC in advance of the intense landfalling Hurricane FRAN in September, 1996 and detailed velocity data were obtained on FRAN's eyewall and can be viewed with latest data from some 1998 landfalling hurricanes on the World Wide Web (address: <http://aaron.ou.edu/dow/>).

A new rugged anemometer wind measurement system with durable supporting 10 m mast has been developed under an Small Business Innovative Research award from NOAA (Richardson et al, 1998). This system has been specified to be both quickly deployable(few hours) and for permanent installations, and capable of measuring and recording windspeeds in excess of 100 m/sec in tropical cyclones and tornadoes. Reinhold has also developed a portable wind tower with two levels of instrumentation, which gathered data in 1998 landfalling hurricanes. It appears that future efforts at obtaining more complete high resolution surface windfield mapping and time-series data in landfalling tropical cyclones and other severe storms could benefit from a pre-storm deployment of these new technologies in the expected path of the storm. Such data would provide invaluable validation of needed post-storm damage assessments and windspeed estimates derived from them, as well as building load data. Recent statistical studies of tornadoes and waterspouts in Japan suggest that on average 20.5 tornadoes and 4.5 waterspouts occur per year in Japan, and 20% of tornadoes are associated with typhoons(Niino et al, 1997). Overall, current estimates of maximum likely peak gusts at standard anemometer height in U. S. hurricanes at landfall are about 80 m/s (ANDREW, 1992) and in 99% of all tornadoes less than 95-115 m/s (values of 124 m/s were measured by portable Doppler in one Oklahoma tornado in 1991, but height was unknown-- Bluestein et al, 1995). *Recently in the Oklahoma City, OK tornado outbreak on May 3, 1999, a record windspeed of 146 m/s was measured by Wurman and colleagues with the DOW in the most intense F5 tornado, within 100 m of the surface* (preliminary data, on Web Site).

It may now be possible using the new technologies above and others such as the now-operational network of coastal NEXRAD Doppler radars in the U.S. to produce real-time damage assessment of hurricanes/typhoons approaching landfall. Powell et al(1995) have shown that using their surface wind analysis software and applying it to aircraft reconnaissance measurements as well as all sources of surface winds, can produce detailed analyses of surface windfields in tropical cyclones. During landfall of a tropical cyclone, real-time analyses of measurements taken by reconnaissance aircraft, land, marine and satellite platforms can help to identify communities experiencing the most severe winds and storm surge. Real-time damage assessment is now possible by using Geographic Information Systems (GIS) to link meteorological windfield information to damage statistics from infrastructure databases obtained by wind engineers. Some potential real-time applications of surface wind field products to warning dissemination, emergency management, storm surge and wave forecasting and wind engineering have been given by Powell and Houston(1996a,b) in a landmark study of Hurricane ANDREW's landfall in South Florida.

The wind engineers at Texas Tech University have pioneered in the development of sheltering concepts for occupant protection from extreme wind events such as tornadoes and tropical cyclones. Tornadic-resistant design was thought to be impossible, due to the widespread belief several years ago that tornado wind speeds might exceed 600 mph(Flora, 1973). However, Golden and Snow (1991) have shown that the highest credible windspeeds in tornadoes near the ground *probably do not exceed 135 m/sec*.

Researchers of wind damage have observed that even if an entire home is destroyed, often a small room, such as a bathroom or closet, remain intact. This phenomena is thought to occur due to the room's small size, ie. closer spacing of structural members. The plumbing and fixtures found in bathrooms are additional factors beyond size that probably help the room resist destruction. In many cases documented after significant tornadoes, these small rooms provided a place of safety for the occupants of the building.

These two discoveries provide practical and economic guidelines for occupant safety from tornadoes and tropical cyclones(though perhaps not at immediate coastal sites subject to storm surges). Wind engineers at Texas Tech reasoned that if one small interior room could be engineered to provide the strength to resist the wind forces and wind generated missiles of a wind event, then this area could be considered a reliable place of safety for the occupants of the residence, i.e. an in-residence shelter. Economical designs and plans for constructing in-residence shelters for the average U.S. home-owner, including guidelines for retro-fitting older homes, have been published (Kiesling and Goolsby, 1974). A comprehensive report has recently been published on residential design criteria for above-ground storm shelters in both tornado and hurricane-prone areas of the U.S. (Mehta, 1996). Reinhold and his colleagues at Clemson University are also working on in-residence sheltering for hurricanes, with emphasis on protection from falling trees.

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including guidelines for retro-fitting older homes, have been published (Kiesling and Goolsby, 1974). A comprehensive report has recently been published on residential design criteria for above-ground storm shelters in both tornado and hurricane-prone areas of the U.S. (Mehta, 1996).

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10km and Under 1km Scale Wind Speed Fluctuation Observed in Typhoon Area.

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Abstract

In order to reveal the structure of strong winds associated with Typhoon VIOLET, we analyzed the data observed by the Narita Airport Weather Radar. The typhoon approached Kanto district on September 22, 1996 and brought strong winds and heavy rain. Ten-km-scale linear echoes were observed north of the typhoon center. The linear echoes had an orientation of NNE to SSW and associated with a large Doppler velocity variation of 5 to 10 m/s. They moved from west to east. In addition, the wind speed fluctuation of the scale less than 1km was also observed. Such large fluctuations of wind speed will affect the detailed prediction of wind and the architecture design about wind endurance.

1 INTRODUCTION

Typhoon VIOLET had approached Kanto district nearest, and caused heaviest damage in recent years.

1996, the second operational doppler RADAR in Japan was deployed (the following Narita radar) at New Tokyo Aviation Local Meteorological Observatory (NARITA Airport). Typhoon VIOLET was the first typhoon approached the Narita radar.

Since ,in Japan, there was no operational instrment that could catch wind speed in the high density spatially and time-related under the strong wind like the typhoon, the data of the Narita radar is precious. The author is investigating the mesoscale phenomenon in the typhoon, and , as the initial stage, analyzed the data of Narita radar . As the result, in the typhoon VIOLET the 10km scale raind band was found.

2 DATA ACQUISITION

The Narita radar has the high resolution that is 0.7 degree in azimuth direction and 150m in ruidius direction, within low elevation, has the time resolution of about 1 minute and 20 seconds. The maximum range of detection is 120km.

Since the Narita radar caught the most useful data of the typhoon VIOLET from 13 to 17 o'clock 22nd, the data of this time was used for analysis.

3 RESULT OF ANALYSIS 1 — 10KM SCALE FLUCTUATION

As the first stage of the analyses, the time sequence of the radar reflectivity and the doppler velocity was made. At this time, the reflectivity and the doppler velocity were superimposed for seeing the relation of position of each other. As the region of analysis, Ibaraki prefecture and its east open sea were took note of.

Indicated by the solid air-line in the figure 1, in this region, the several linear parts of the strong reflectivity which run from NNE to SSW paralleled.

Then, it is found that the contour of doppler velocity was lifted up along the rainband. In this region, the reflectivity and doppler velocity ranged wavelly, and the phases of both were synchronized.

The distribution of both like this occurred from 13 to 16 JST during the analysis time, it moved continuously from west to east keeping the synchronization of the phase like the propagation of wave.

Before 15 JST, the interval between rainbands was about 10km, from 15 to 16 JST, that was about 20km. The amplitude of the fluctuation of the doppler velocity was 5 ~ 8m/s, the moving speed of the echoes was about 30 m/s.

The figure 2 shows the vertical cross section on the circle of the 80 km radius from the Narita radar. The structure of the synchronization of the fluctuation of the reflectivity and the doppler velocity reaches the altitude 5 ~ 6 km from low level altitude.

4 RESULT OF ANALYSIS 2 — UNDER 1 KM SCALE FLUCTUATION

The figure 3 shows the reflectivity and the doppler velocity in the area within 5km east, west, north and south each direction from the Narita radar. The striping was found in the figure of the doppler velocity. The stripes run NE to SW, and the interval of stripes is less than 1 km. There is the wave which has wavelength less than 1 km in the doppler velocity field. The amplitude of the wave is about 5 m/s. On the other hand, the reflectivity has no such fluctuation.

A comparison between the doppler velocity of 6 elevations from 0.7 degree to 17.0 degree indicates that the wave reached to only the altitude lower than 1km from the ground level.

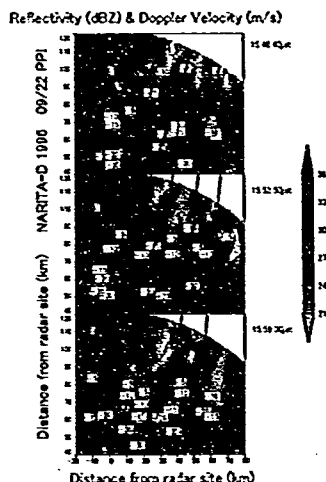


Figure 1: The reflectivity(shade: dBZ) and the doppler velocity (broken line: component approaching to the radar site thin solid line: component away from site m/s) of Narita radar at 15:46JST, 15:52JST and 15:59JST, 22nd. The thick solid straight lines indicate the location of the remarked rainbands. The thin solid straight lines indicate the motion of the rainbands.

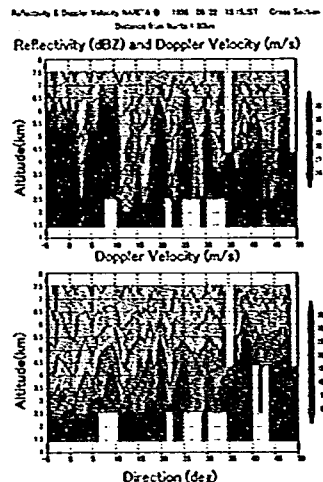


Figure 2: The vertical cross section at 15:15JST, 22nd on the circle whose radius is 80 km from Narita radar. Upper figure indicates the reflectivity(shade: dBZ) and the doppler velocity(contour: m/s). Lower indicates the doppler velocity(shade: m/s). The axis of abscissas indicates the azimuth (north is zero), and the one of ordinates indicates the altitude.

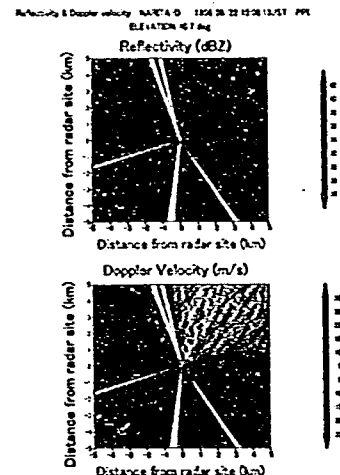


Figure 3: The reflectivity(upper: dBZ) and the doppler velocity(lower: m/s) near of Narita radar at 15:08JST, 22nd. The striping which runs from NE to SW was found in the figure of the doppler velocity. There was not anything correspondent with it in the reflectivity.

5 EXAMINATION

5.1 10 KM SCALE FLUCTUATION

Gall. et al(1998) observed 10 km scale rainbands near the eye wall. They considered that it was same as the roll convection in the boundary layer and not the gravity wave.

When the lower of the boundary layer having no wind becomes unstable *Reyleigh-Bénard* convection occurs. But if the wind having vertical shear blows, it is structured to the horizontal roll vortex. Usually, the roll vortex in the boundary layer has 2 ~ 20 km wave length and 1 ~ 2 km vertical width. Since the rainband observed by them was the convection having 5 ~ 6 km height, it was taller than the usuall roll convection. They suggested that it was because the unstable layer of the atmosphere was much deep.

It is not known whether the 10 km scale rainband of typhoon VIOLET described at the section 3 is the roll convection or the gravity wave, because the analysis has not proceeded yet. At all events, this rainband had the structure of the much wind speed

change or the much direction change, and it is considered that it is not a little important for the prediction and the estimation of the effect of the wind of typhoon.

5.2 UNDER 1 KM SCALE FLUCTUATION

Wurman and Winslow(1998) got the result of the observation like the typhoon VIOLET in the hurricane Fran, and considered that it was because the wind structure was same as the one of the roll convection in boundary layer. Although the roll convection in boundary layer usually occurs in the convective unstable layer, because of the obscurely unstable layer in thier observation they could not manifest the origin of the phenomenon.

Since the phenomenon of typhoon VIOLET was same as the thier observation with regard to both of the horizontal scale and the vertical distribution, perhaps, it had the same wind structure with the roll convection in the boundary layer.

6 CONCLUSION

Tow scale wave like fluctuations were observed on the typhoon VIOLET.

10 km scale fluctuation had 5 ~ 10 m/s amplitude of doppler velocity and 10 ~ 20 km wavelength. This phenomenon continued during more than three hours when the typhoon approached in Kanto district at closest range, and it had 5 ~ 6 km height. The typhoon had the mesoscale precipitation system which had the structure that the fluctuation of wind and rainfall synchronized.

Under 1 km scale fluctuation had under 1 km wavelength and under 1 km height.

7 ACKNOWLEDGEMENT

Data used in this study provided by New Tokyo Aviation Local Meteorological Observatory.

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Understanding Wind Speeds in Hurricanes: Implications for Meteorological and Engineering Applications

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Abstract

Significant new technologies are now being employed in the US to study the structure of hurricane wind fields. The technologies include GPS dropwindsondes, fixed, portable, and airborne Doppler radars, wind profilers, deployable trailers with responsive propeller anemometry, and new airborne passive microwave sensors. These new technologies offer the potential for significant improvements in the quantification and characterization of the wind structure within hurricanes. The warning and forecast problem is discussed from the engineering and atmospheric science perspectives and we present examples of a few new wind measurement technologies.

INTRODUCTION

In engineering applications, design usually is directed at typical cases of extreme events and very rare events are hopefully covered to some extent in the safety factors assumed in working stress design or in the load and resistance factors applied in reliability based design. In contrast to this pragmatic balancing of construction costs, risks, and benefits, warnings based on imperfect forecasts tend to be aimed at the worst conditions which may occur anywhere within the warning zone. Because of uncertainty on the current and predicted intensity of the storm, its future position, and the size of its wind field, a safety factor is built into the warnings such that the magnitude of the event being warned for is often more than what actually occurs and the length of coastline warned is often a factor of 3-4 larger than the area actually affected by the storm. The danger is that people with property which is unaffected by the most extreme events will become convinced that they have in fact weathered the worst conditions and not take future warnings seriously.

Reducing this overwarning problem is a formidable problem. While forecast motion (position and track) errors have been decreasing over the past 25 years at a rate of about 2% per year for 24, 48 and 73 h periods (McAdie and Lawrence 1999), intensity forecasts show only marginal skill (DeMaria 1998) and actual landfall time and position forecasts only show significant improvement trends for the 30-54 h period before forecast landfall (Powell and Aberson 1999). Hence only slow forecast-related improvements are expected in the future. The warning factors expected to improve most rapidly are those related to

reducing the uncertainty of the current storm intensity and the size of the wind field. An added benefit in this area is the ability to accurately inform residents of the affected areas about the magnitude of the event they have experienced. This paper provides a starting point for further discussion of the issues facing meteorologists and engineers as we seek to both warn people of possible dangers while encouraging them to take action to reduce their vulnerability before the event strikes.

SOME NEW WIND MEASUREMENT TECHNIQUES

Aircraft Remote Wind Sensing in Hurricanes

The NOAA P3 research aircraft has been evaluating the Stepped-Frequency Microwave Radiometer (SFMR) over the past several years with encouraging results. This instrument is a passive microwave system that detects the radiation emitted from bubbles on the sea surface. A well defined relationship between the emissivity of the sea surface and wind speed (Fig. 1) makes possible an algorithm to determine surface wind speed. A plot of flight level and surface wind speeds vs radial distance from the storm center of Hurricane Georges of 1998 (Fig. 2) shows the value of these data. The wind maxima at the surface are radially inward from those at the flight-level and the surface circulation center (as defined by the wind speed minimum) is located in a different location from the flight-level center. The ability to assess surface wind speeds without having to invoke PBL models to adjust flight-level winds to the surface will greatly improve real-time hurricane surface wind field analyses conducted by NOAA-HRD (see storm atlas web page at www.aoml.noaa.gov/hrd). Eventually it is hoped that this instrument may be carried by operational hurricane reconnaissance aircraft operated by the U.S. Air Force Reserves.

Doppler Radar techniques

Application of the Velocity Azimuth Display technique and examination of lowest tilt Doppler radial velocity fields in landfalling hurricanes within range of landbased Doppler radars (Fig. 3) has provided valuable information on wind profiles and characteristic small scale features in the wind field. A logarithmic wind profile is noted within the boundary layer and the small directional shear within the lowest couple of hundred meters suggest the presence of a surface layer. Radial velocity fields examined at the highest resolution show evidence of linear velocity features aligned with the low level wind velocity.

GPS Sondes

Over 100 GPS sondes have now been deployed into hurricane eyewalls including intense Category 4 and 5 storms on the Saffir-Simpson scale. An example of an eyewall velocity profile in Fig. 4 shows the presence of multiple wind maxima in the lowest few hundred meters. The persistence of these features in the most intense storms requires examination of their possible affect on wind load provisions contained in building codes. At this point in time the meteorological and engineering communities are struggling to assimilate this new information in coherent and rational ways for operational functions such as warnings and for a clearer understanding of the practical implications of the results.

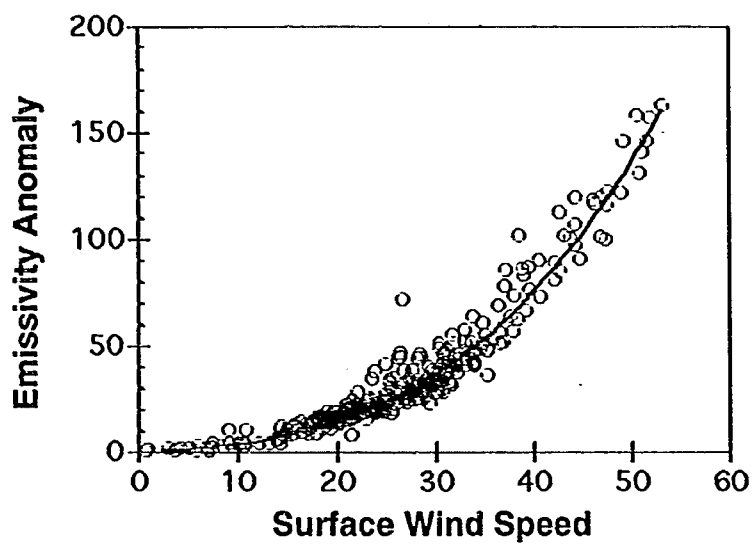


Fig. 1 Relationship between emissivity anomaly and wind speed using SFMR data collected in Hurricane Gilbert (1988).

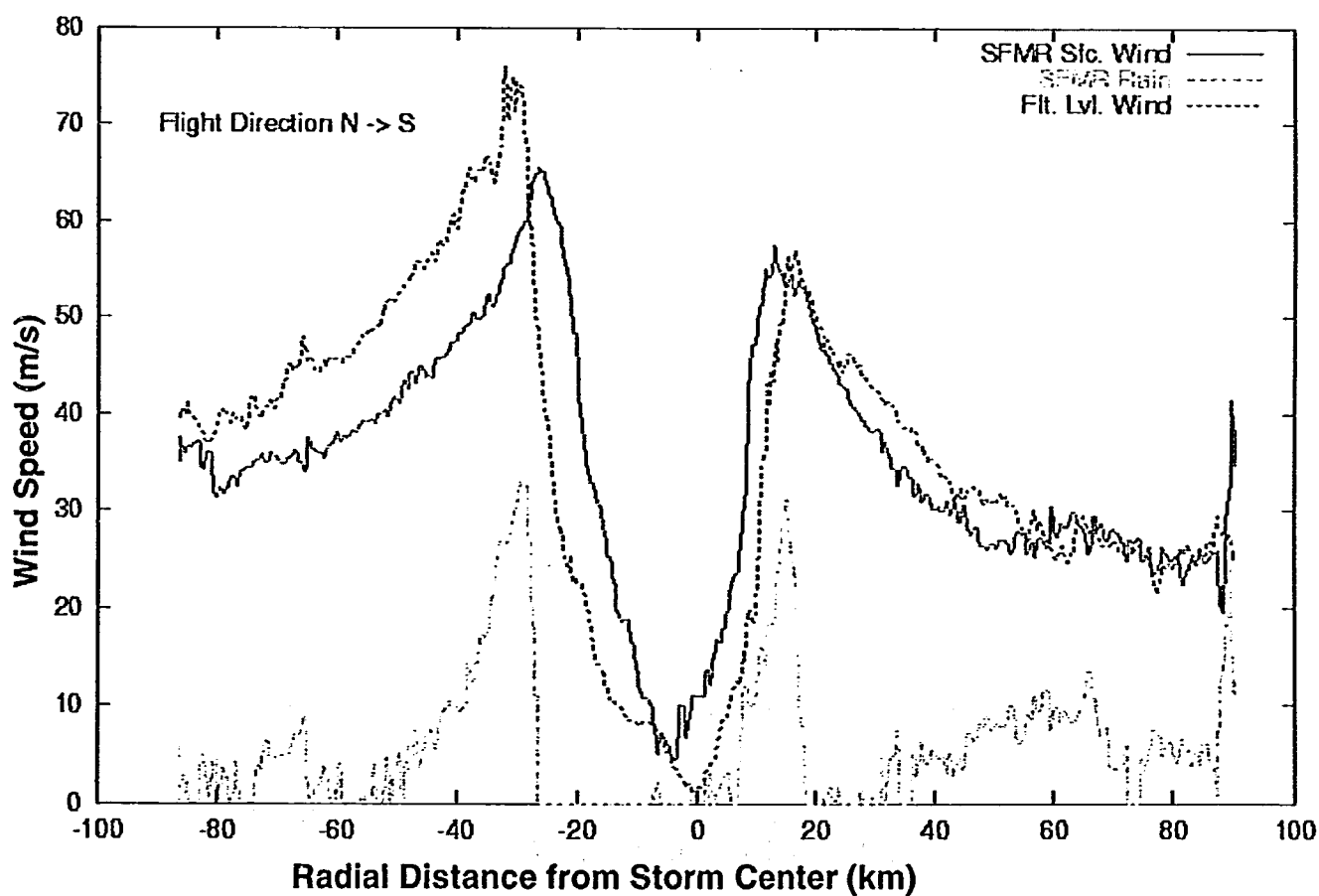


Fig. 2 Radial profile of flight-level (dotted) and SFMR (solid) surface wind speeds, and rain rate in Hurricane Georges(1999)

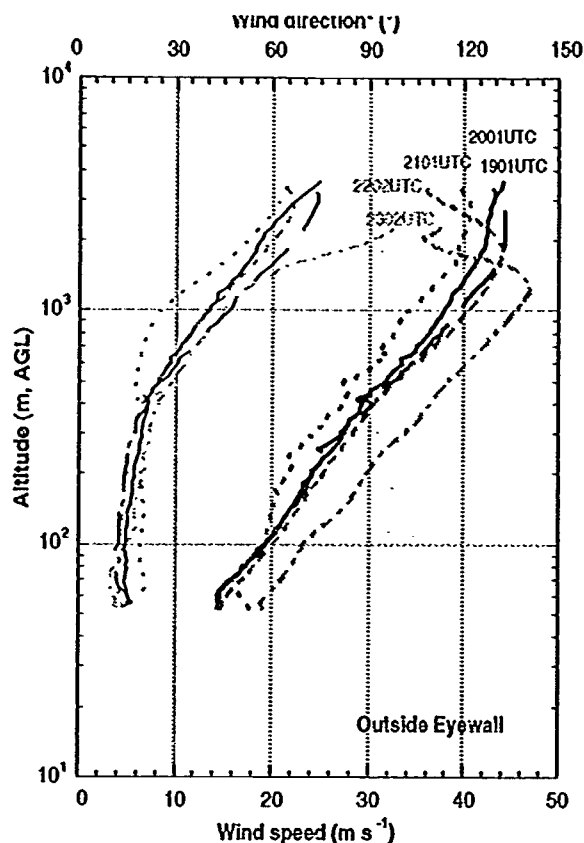


Fig. 3 Doppler radar VAD wind profiles from Wilmington, NC in Hurricane Fran (1996)

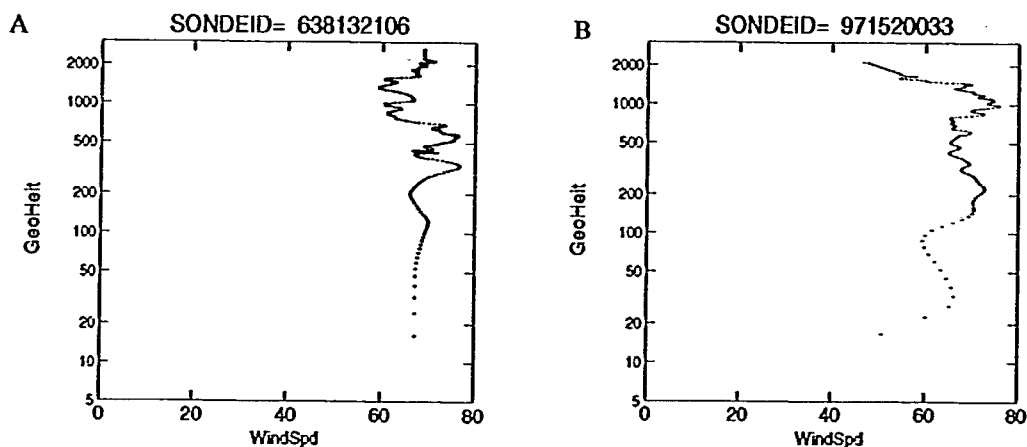


Fig. 4 Eyewall wind speed (m s^{-1}) profiles in A) Guillermo 8-3-97 and B) Hurricane Erika 9-8-97.

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A STUDY ON SPATIAL CORRELATION OF NATURAL WIND

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ABSTRACT

Spatial correlation is one of the important features of wind for gust responses of long span bridges. To study on spatial correlation of natural wind, field measurement of natural wind and related gust responses analysis were conducted. As the result, in low frequency range, it was found that the measured spatial correlation of strong wind was lower than one used in wind-resistant design. As the result, gust responses analysis using measured spatial correlation showed lower amplitude of responses than result from analysis using spatial correlation clarified in design manual. This implies a possibility of decreasing design wind load for long span bridges.

1. INTRODUCTION

In general, the strength of wind load is determined considering the effect of gust responses of structures. Spatial correlation is one of the characteristics of wind which affect gust responses. In wind-resistant design of bridges, spatial correlation of wind clarified in design manual from field measured data is used. However, data on spatial correlation of wind in low frequency range which is very important in gust responses of structures which have low natural frequency, e.g. long span bridges, is not sufficient. Therefore, natural wind was measured at the Ohnaruto bridge, a suspension bridge, and the characteristics of the data were evaluated through gust responses analysis.

2. FIELD MEASUREMENT OF NATURAL WIND

2.1 Measurement System

The layout of anemometers used for this measurement is shown in Figuer-1. Five ultrasonic anemometers installed at top of lighting pole were used, and wind speed data were recorded with sampling frequency of 20Hz. The anemometers were about 74m above sea surface.

2.2 Characteristics of Natural Wind

During measurement period, strong wind due to two typhoons were recorded. The highest record of each typhoon is shown in Table-1. In this report, data due to typhoon9612 is analyzed. The direction of the strong wind analyzed was SSE which is almost

perpendicular to bridge axis(see figure-1), and as the anemometers were installed at the southern side of the girder, the wind are considered to be not affected by girder strongly.

Figure-2 shows a example of the wind data from the typhoon 9612. The intensity of turbulence is about 8%. The power spectrum of this data is shown in Figure-3. As shown in figure-3, the agreement between the power spectrum of measured data and one specified in design manual (Hino's spectrum) is rather good.

In figure-4, as the spatial correlation of wind, co-coherence calculated from measured data is shown. In the figure, spatial correlation (co-coherence) clarified in design manual (Equation-1) is also shown. As shown in the figure-4, spatial correlation calculated from the measured data is high when the distance is short, and frequency is low. However, the spatial correlation calculated from the measured data is lower than equation (1), which is used in design, especially in low frequency range.

$$\text{Co-coherence} = \exp \left(-k \frac{f \cdot \Delta y}{U} \right) \quad (1)$$

where k:decay factor, f:frequency,
 Δy :transverse distance, U:wind speed

3.GUST RESPONSES ANALYSIS

3.1 Analytical Conditions

In order to evaluate the effect of the difference of spatial correlation between measured data and one clarified in design manual, gust responses analysis was conducted. Major conditions of two cases of gust responses analysis are shown in Table-2. The only difference between the two cases is in spatial correlation. Objective of this analysis is the Ohnaruto Bridge.

3.2 Result of Analysis

As the result of analysis, responses in horizontal component are shown in Table-3. Due to the difference in spatial correlation, maximum amplitude of vibration in case-2 is about 70% of case-1. As for the maximum displacement, which is sum of static displacement and amplitude of vibration, case-2 is about 90% of case-1. This means if spatial correlation from measured data can be used in design, strength of wind load can be decreased from present wind load strength.

4.CONCLUSION

Spatial correlation calculated from field measurement of natural wind was lower than one clarified in wind resistant manual. As the result, it was shown through gust responses analysis that strength of wind load can be decreased if spatial correlation from measured data

can be used in design. However, in order to change design condition, it is necessary to collect more data on natural wind, and understand its characteristics.

References:

- 1) Honshu-Shikoku Bridge Authority(1990). *Wind design manual for the Akashi-Kaikyo Bridge*, Honshu-Shikoku Bridge Authority, JAPAN

Table 1. Recorded Strong Wind

Date	Weather Condition	Measured Location	Wind Speed * m/s
1996/08/14	Typhoon 9612	Ohnaruto Bridge	35.5
1996/09/22	Typhoon 9617	Ohnaruto Bridge	20.9

*: Average of 10 min.

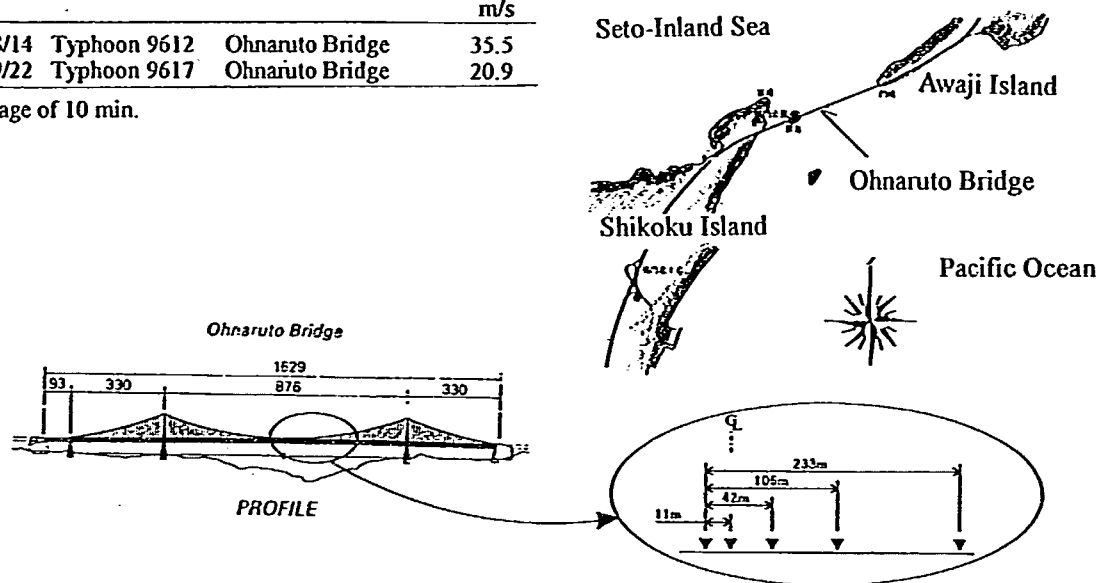


Figure-1 Layout of Anemometers

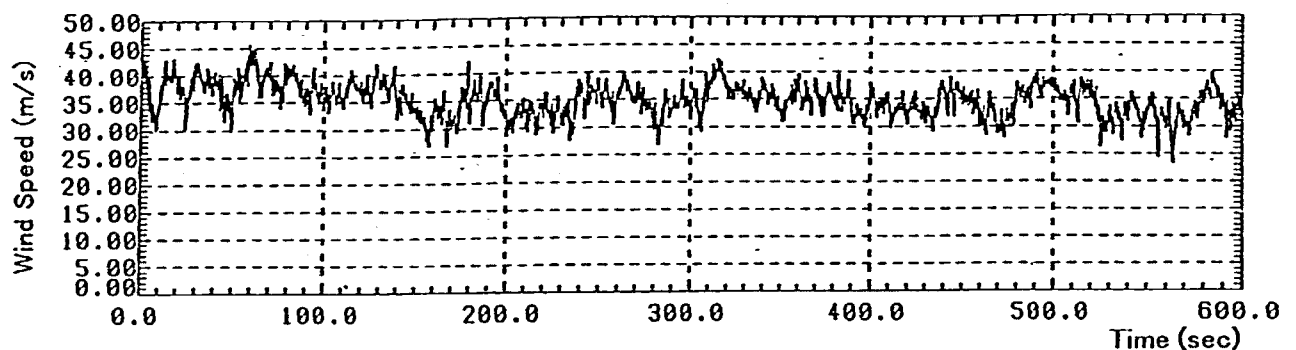


Figure-2 Strong Wind Data by Typhoon 9612

Typhoon Wind Data

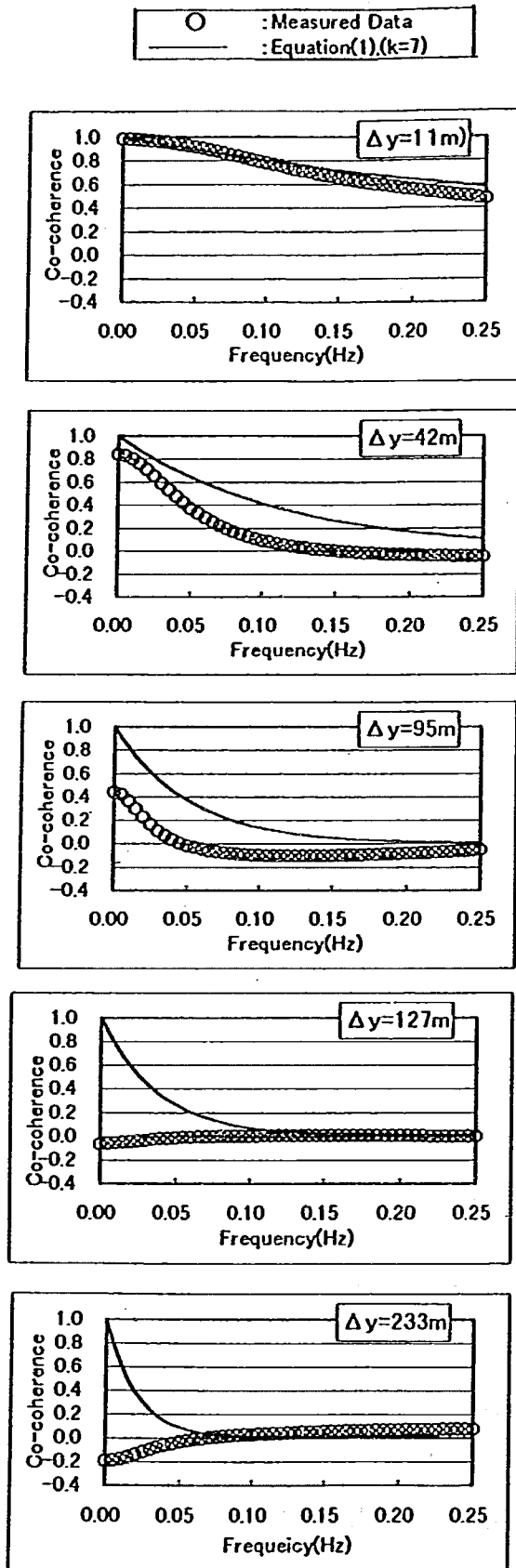


Figure-4 Spatial Correlation

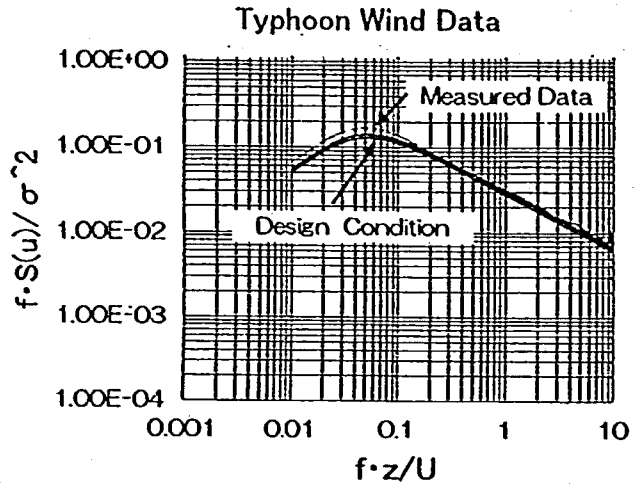


Figure-3 Power Spectrum

Table 2. Gust Response Analysis Condition (Ohnaruto Bridge)

Item	Case-1	Case-2
Spatial Correlation	Design Condition*	Measured Data
Power Spectrum	Measured Data	Measured Data
Turbulence Intensity	0.09	0.09
Wind Speed **	34.3 m/s	34.3 m/s
Air Density	$0.125 \text{ kgf} \cdot \text{s}^2/\text{m}^4$	$0.125 \text{ kgf} \cdot \text{s}^2/\text{m}^4$
Structural Damping***	0.10****	0.10****
Modes Considered	1-100 th mode	1-100 th mode

* : Based on wind resistant design manual for

Akashi-Kaikyo Bridge (Ref. 1)

** : Wind Speed at height of girder, average of 10 min.

*** : 1st Symmetric Bending Mode

**** : Vibration Test Result

Other Analytical Conditions are based on Ref. 1

Table 3. Results of Gust Response Analysis (Ohnaruto Bridge)

Item	Case-1	Case-2
Static Displacement	150 cm	150 cm
Max. Vibrational Amplitude	71 cm	48 cm
Max. Displacement	221 cm	198 cm

Horizontal Component

A Modeling Study of the Development of Mesoscale Asymmetries in Tropical Cyclone Structure During Landfall

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ABSTRACT: Mesoscale models have been developed to simulate the multi-scale interactions that occur in the atmosphere, which are necessary for successful numerical weather prediction of fine-scale weather phenomena. The U.S. Navy Coupled Ocean/Atmosphere Model Prediction System is currently capable of nesting down to fine resolutions limited only by the resolution of the orographic database and the accuracy of the physics packages employed. Here, a modeling study of the landfall of Typhoon Peter over Japan is used to illustrate the complex modification of the low-level wind and precipitation fields that can be resolved by COAMPS as the typhoon interacts with the steep topography of the Japanese islands.

INTRODUCTION

The interaction and modification of a tropical cyclone as it encounters relatively flat land has been studied in some detail (e.g., Tuleya et al. 1984). Low-level wind, moisture, and precipitation fields become noticeably asymmetric with respect to the coastline at landfall. Rapid filling of the storm generally occurs several hours after landing, primarily due to the suppression of the surface moisture flux. As a tropical cyclone encounters a coast with mountains, more complex changes in the structure may be expected (e.g., Bender et al. 1987). Modification to the wind field as it interacts with significant orography is expected to produce localized forcing of vertical motion and clouds that are hypothesized to force local extremes in precipitation. These asymmetric and localized wind and precipitation patterns are one of the main forecasting challenges as a tropical cyclone makes landfall.

TY Peter formed in the western North Pacific during late June 1997 (Fig. 1). The deep-layer mean flow between an upper-level trough over the Asian mainland and a strong anticyclone to the southeast of Japan produced an initially northward track. By 12 UTC 27 June, the mid-latitude trough had propagated to a position north of the storm and the 500-mb geopotential height perturbation associated with Peter was located at the base of this trough. At this time, the storm began to move in an east-northeast direction toward Japan (Fig. 1).

As TY Peter approached Japan, transformation to an extratropical cyclone became evident in satellite infrared and SSM/I imagery. Whereas the deep convection was suppressed in the southern core region, the outflow aloft enhanced by interaction with the mid-latitude westerlies resulted in strong convection around the north side of the core that was associated with enhanced precipitation at the southern end of Kyūshū Island. Just prior to landfall in Japan, TY Peter connected with the mid-latitude trough that had moved to its north. The mid- to upper-level circulation of Peter merged with the trough that produced an apparent southward extension of the trough. As the system traveled along the main island of Honshu, it gradually weakened from 984 to 990 mb. However, an elongated band of convection and precipitation was present over the Japan Sea, north of Honshu throughout the 24-h passage of the storm. As the system passed off the islands on 29 June, it completed extratropical transition as it coupled strongly with the mid-latitude trough, and rapidly re-intensified to 980 mb.

METHODOLOGY

Model description

To properly investigate topographic effects on storm structure, it is necessary to use a modeling system capable of resolving not only the dynamic features of the storm, but also the orography that is expected to force low-level asymmetries. The atmospheric model employed in this study is the U.S. Navy Coupled Ocean/Atmosphere Model Prediction System (COAMPS) described in detail by Hodur (1997). However, an ocean prediction model is not included in this study. The atmospheric system is nonhydrostatic, has three nested grids, high resolution (1 km) topographic databases, and includes a Kain-Fritsch (K-F) representation of convection, explicit moist physics, and boundary layer processes.

To represent both the large- and mesoscale processes that were important for the transition of TY Peter into an extratropical system, three meshes were employed with resolutions of 81, 27, and 9 km for the coarse (CM), middle (MM), and fine (FM) meshes, respectively, on a Lambert conformal projection. Whereas the CM domain of 7400 km x 5100 km was large enough to allow adequate representation of upstream large-scale features of interest during the integration, the FM domain of 1220 km x 1200 km allowed a more detailed specification of the mountainous terrain of the Japanese islands. On the CM and MM, the K-F scheme is employed to treat sub-grid-scale convective processes, while the explicit moist physics is used to treat the grid-scale saturation. On the FM, only explicit moist physics is employed.

The MM and FM have two-way interaction, with all variables calculated on the FM being passed to the MM at the end of each MM time step, and all variables on the MM being passed to the CM after each CM time step. Information from the outer meshes is passed to the inner meshes via the boundaries. Boundary conditions for the CM are provided from global analyses.

Analysis fields

High-resolution (1° lat./long.) analyses from the Naval Operational Global Atmospheric Prediction System (NOGAPS) provide the background fields, which are interpolated down to the 81, 27, and 9 km grids. These NOGAPS analyses, which incorporate high-density water vapor and visible satellite cloud-drift winds as well as all available conventional data sources, have been shown in various studies to satisfactorily resolve the large-scale features in the western North Pacific (e.g., Harr et al. 1996). A re-analysis step is used to analyze additional observations on the MM and FM so that mesoscale details are incorporated into the model fields at the initial time.

The model was initialized at 12 UTC 27 June and a 48-h forecast was performed.

RESULTS

COAMPS 48-h simulation

Storm track The storm has an initial central pressure of 984 mb near 30°N, 127°E. By 24 h, it has a central pressure of 987 mb. Although it is following the same path as TY Peter, the predicted positions are about 2 h slow. The system moves off the coast at 00 UTC 29 June with a pressure of 990 mb and continues to weaken in the final 12 h of the simulation.

Precipitation patterns In the first 24 h, a maximum of 19 cm of precipitation occurs on the

southern end of Kyūshū. This is less than the 25 cm in the verifying NOGAPS analyses, which is not a surprising result given that some spin-up of the convective scheme takes place in the first 6 h of the simulation. The K-F scheme also has a tendency to under-forecast precipitation (S. Chen, Naval Research Laboratory - personal communication).

After 12 h, grid-scale saturation is producing most of the precipitation, and almost all of the precipitation occurs to the north of the storm track. The 3-h total precipitation from 21 to 24 h of the simulation matches well with the SSM/I 85 GHz image over the storm near the verifying time of 12 UTC 18 June. The SSM/I 85 GHz channel measures ice particle scattering that is highly correlated with precipitation. The similarity of the two patterns is not coincidental. Interactions between the low-level storm and the topography produce complex flow patterns with localized convergence and upslope motion that results in preferential regions of stable precipitation.

Development of the wind field As the storm interacts with the topography, complex asymmetries in the low-level wind field develop after 24 h. As the southerly flow approaches the island, upslope motion occurs along the mountain ridge on the south side of the island. At midlevels, these vertical velocities are capped by weak subsidence. Downslope motion is predicted on the northern side of the island and the subsequent offshore flow converges with a larger scale northerly flow toward the storm. Although the main precipitation pattern might be expected to develop on the southern side of the island where the upslope motion is strongest, the maximum precipitation actually occurs (in the model) off the north shore where the low-level convergence is the strongest, and this is consistent with the rain rates in the 85 GHz SSM/I imagery.

At mid levels, the storm core weakens and is absorbed into the mid-latitude trough. The under-estimation of convective precipitation by the K-F scheme noted earlier might result in more weakening of the storm at these levels than the verifying analyses indicate. The southward extension of the midlatitude trough was hypothesized earlier to be a necessary component for the strong coupling with the tropical cyclone. Since the southward extension of the trough depends on merger with the mid-level storm core, such an over-weakening of the storm may explain why the trough did not extend as strongly equatorward as in the analyses. Thus, there was only a weak coupling between the storm and the midlatitude trough that led to an under-forecast of the subsequent re-intensification of the extratropical system.

CONCLUSIONS

An analysis and simulation of Typhoon Peter that passed over Japan during transition to an extratropical system has been presented. The path of the storm is well represented in the model simulation, although the positions lag by about 2 h. However, the coupling between the remnants of TY Peter and the midlatitude trough that is necessary for the observed strong re-intensification as it passed off the Japan islands is not sufficient in the simulation. In addition, the low-level storm structure is highly disrupted by interaction with the island topography. Strong, localized asymmetries in the wind field, and resulting vertical velocity patterns over Honshu, modulate precipitation processes in a manner similar to that observed in IR and SSM/I imagery for corresponding times. Precipitation mainly occurs to the north of the storm track (western quadrant) as observed in this imagery. As the storm interacts with the topography, complex flow patterns develop even at a 9-km resolution. Although these simulations could be made at as fine a resolution as 1 km, matching the resolution of the topographic database, detailed automated station data and radar data over Japan are vital to verifying the validity of the model simulations

before further progress is warranted.

Acknowledgments: This research is funded by the U.S. Office of Naval Research Marine Meteorology Program.

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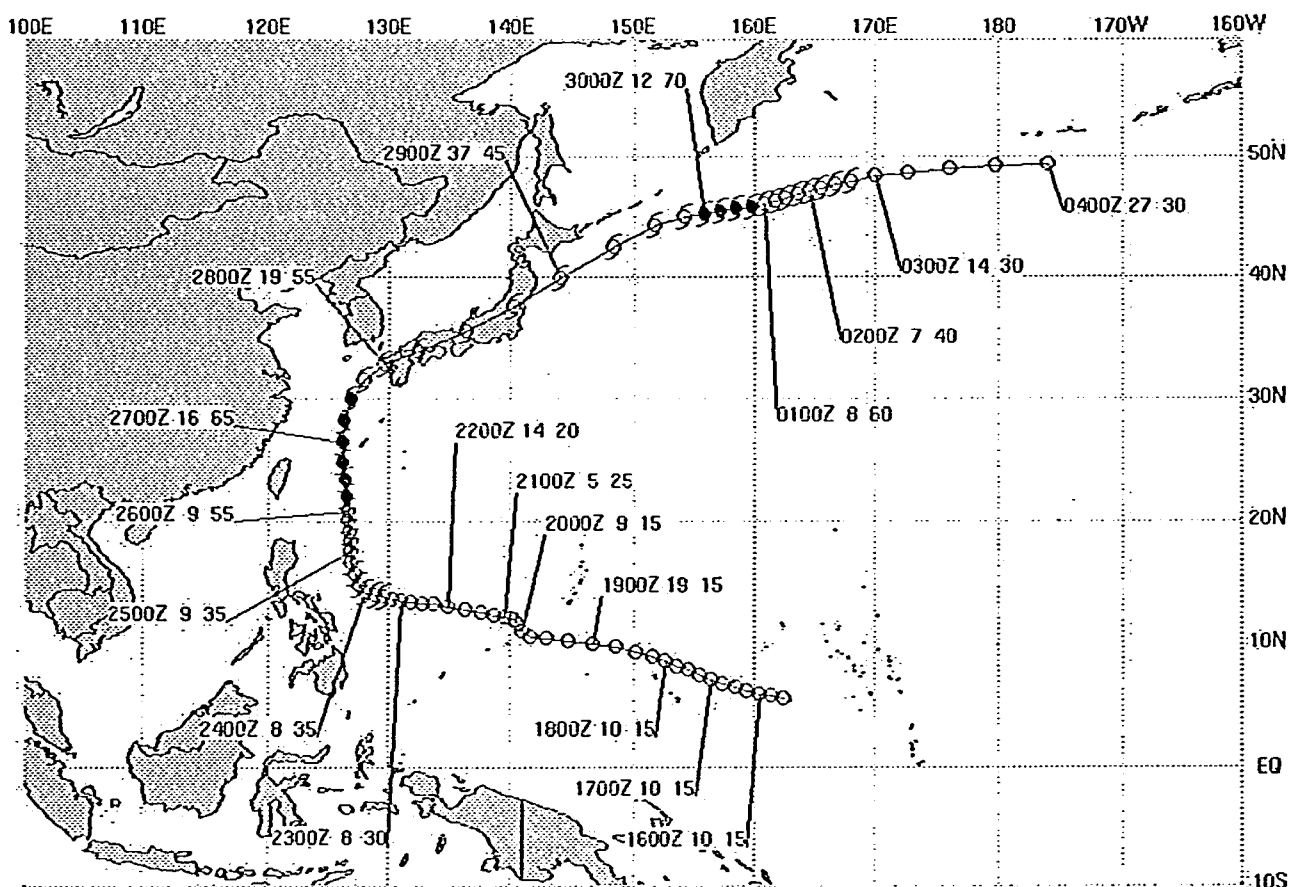
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Fig. 1: Six-hourly best track positions for Typhoon Peter. Period of interest begins at 12 UTC 27 June as Peter begins to turn toward Japan and continues until 12 UTC 29 June.



Session 2 Wind Flow, Pressure and Loading

(Chaired by A. Chiu)

- (1) Comparison of wind pressure measurements on a tower-like structure between full-scale observation and wind tunnel test.

(H.Okada, K. Yasui, K. Yoshikawa, M. Watakabe, M. Okabe and H. Kikitsu)

- (2) Roof corner wind flow and pressures.

(P. Sarkar, D. Smith and K. Mehta)

- (3) Extreme load distributions on buildings with a rectangular plan.

(Y. Tamura)

- (4) Performance-based objective design and natural disaster risk management.

(G. Chiu)

- (5) Feasibility of CFD technique for prediction of wind flows and wind loading.

(T. Tamura)

- (6) Towards refined modeling of wind effects through collaborative research.

(B. Bienkiewicz)

Comparison of wind pressure measurements on a tower-like structure between full-scale observation and wind tunnel test

Hisashi Okada*1, Kenji Yasui *2, Kiyomine Yoshikawa*3,
Morimasa Watakabe*4, Minoru Okabe*5, Hitomitsu Kikitsu*1

1. Introduction

A research project on "Building Structural Design based on Wind Tunnel Tests" has been carried out since 1997. This research project is carried out as a cooperative research supported by BRIC that is Tsukuba Building Research Institute Community composed of BRI, non-profit Agency, and private company's research institutes in Tsukuba area. The members participating in this project are listed in Appendix. In the project, comparisons of dynamic behavior of a tower-like structure and pressures on walls of the structure between full-scale measurements and wind tunnel tests have been carried out. In this paper, the comparison of pressure measurements is described. Objective of the comparison is a survey on the applicability of wind pressure coefficients obtained in the wind tunnel tests.

2. Full-scale pressure measurement

2.1 Tower-like structure

Figure 1 shows the structure on which pressure measurement described here has been carried out. The structure was constructed for the tests on passive and active vibration control systems in the Technical Research Institute, Okumura Cooperation in 1990. Since the completion, observation of dynamic response to wind excitation has been carried out. Wind pressure measurements started in 1996. Height of the structure is 31m. Its 5th and 6th stories have external walls of ALC panels, but stories from 1st to 4th do not have external walls but only floors and posts. Pressure measurement points are set on the external walls of 5th and 6th stories.

There are no obstructions around the site, which may have serious influence to the approaching winds.

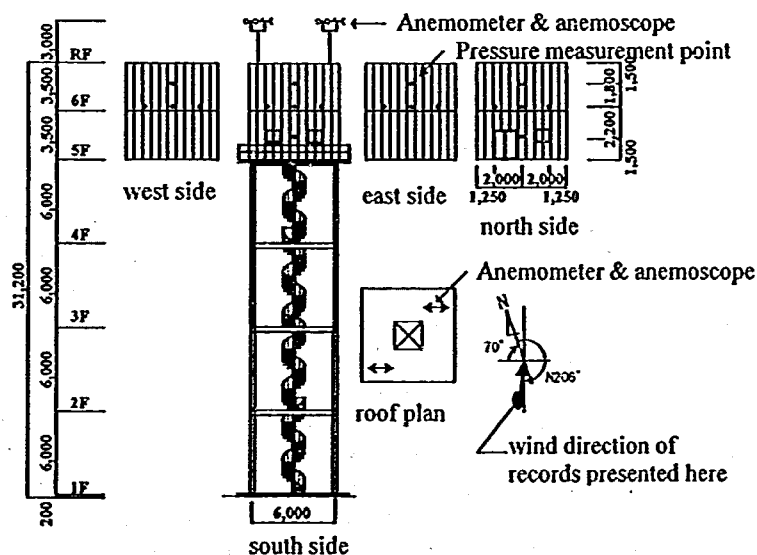


Figure 1 Tower-like structure for pressure measurement.

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*2 Technical Research Institute, Okumura Corporation, *3 Technical Research Institute, Tobishima Corporation,

*4 Technical Research Institute, Toda Corporation, *5 Center for Better Living, Tsukuba Building Test Laboratory

2.2 Pressure measurement

Pressure measurement points are arranged on the 5th and 6th external walls as shown in Figure 1. They are arranged in cross shape on the north and south walls and in shape of upside T on the east and west walls. The numbers are 5 on north and south, 4 on east and west, respectively. Figure 2 shows the pressure measurement system used in the observation. Wind pressures on the pressure measurement points are led to measurement pressure ports of wind pressure gages. To the reference pressure ports of wind pressure gages, a constant pressure is distributed as reference pressure. The constant pressure is supplied through a pressure control valves from a nitrogen cylinder and its level is atmospheric pressure at the time of initial setting +2.8 kilo Pascal. The difference pressure between two pressures can be measured by the pressure gages.

At the time of pressure measurement, the measurement port of the upper pressure gage on the west or east wall was opened to the room in order to measure the internal pressure. It should be noted that there are little openings on the walls of 5th and 6th stories and roof but floors have openings for the stair. The conditions that are positions, areas and etc., define the internal pressure.

Wind anemometers and anemoscope are installed on 3 m height poles, which are mounted at the north-east and south-west corners of the roof. The anemometer is a three cap type having 5.1 m distance constant, and anemoscope is a arrow shape type and its distance constant 3.5m. Pressure data were sampled with 10 Hz and accumulated as data having 10 min. length composed of 6,000 continuous data.

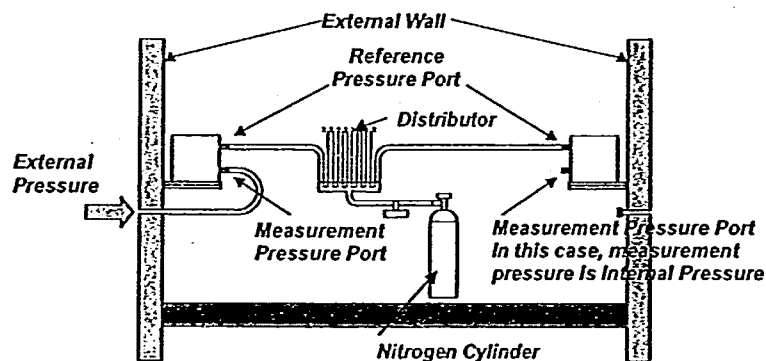


Figure 2 Pressure measurement system

3. Wind tunnel tests

Wind tunnel tests were carried out in the Boundary Layer Wind Tunnel of Building Research Institute, Ministry of Construction. In the tunnel, natural wind was simulated. The target natural wind was the one over the terrain of category III of AIJ recommendation for suburban area. The simulated wind has almost the same mean wind velocity profile as the target one, but its turbulence is smaller and the intensity at the top of the structure is 9% while the target intensity 20%.

Figure 3 shows a model of the structure used in the wind tunnel tests. The length scale is 1/60, from the consideration of its blockage ratio to the wind tunnel test section. The positions of pressure taps are the same positions of full-scale plus points on corners of the walls, on the roof and on the bottom. The total number of pressure taps is 53 ; that is 9 on each walls and roof and 8 on bottom. The pressures on the taps are led to pressure transducers by vinyl tube of 1,000mm length. Static pressure in wind tunnel is used as reference pressure for the pressure measurement. Influence of the tube to the pressure measurement is taken away by Fourier transformation technique, using the dynamic pressure transferring property. The reference dynamic pressure is measured with pitot pipe set at the position of 1,000mm height and 1.5 m windward side of the model.

Table 1 shows summary on reduction ratios of the model and data sampling conditions. Time ratio is 1/30. Wind velocity ratio is 1/2 which can be obtained from time and length ratios.

Wind direction perpendicular to a wall was selected for the test because those of full scale measurement records obtained were almost perpendicular to the south wall of the structure.

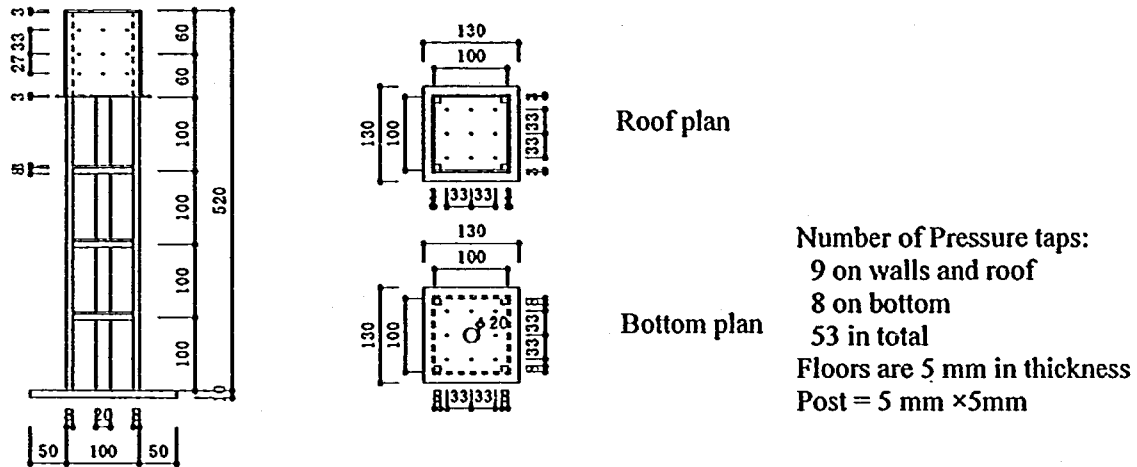


Figure 3 Model of wind tunnel test

Table 1 Wind tunnel measurement conditions

reduction ratio	length	1/60
	time	1/30
	wind velocity	1/2
data sampling frequency		400 (Hz)
data length (number of data)		8192
cut-off frequency of low pass		200 (Hz)

4. Results of full-scale measurements and wind tunnel tests

4.1 Time histories of record in full-scale measurements.

Four records in full-scale measurement were obtained on March 20 in 1998. The time histories of wind direction, velocity and internal pressure of the records are shown in figure 4. The mean wind directions are 6 degree from direction perpendicular to the south surface to west and fairly stable. The mean wind velocities are 15 to 17m/s and the changes are very little.

Their turbulent intensities are about 20%. On the other hand, the internal pressures show a tendency of decreasing lineally. Taking into consideration of this result, the evaluations of the external pressures were made by subtracting the mean internal pressure during the measurement from the measurement ones.

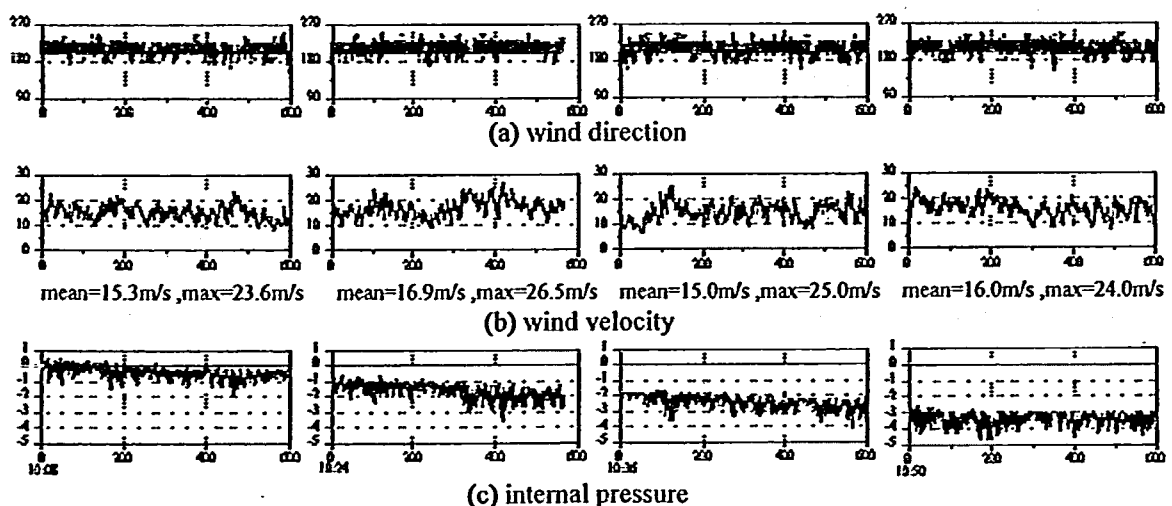


Figure 4 Time histories of wind direction and velocity and internal pressure in full-scale measurements on March 20 in 1998

4.2 External wind pressure coefficients

Mean and rms wind pressure coefficients are defined as follows.

$$C = \frac{P_{mean}}{q_H}, \quad C_{rms} = \frac{P_{rms}}{q_H} \quad (1)$$

where, C, C_{rms} = mean and rms wind pressure coefficient respectively.

P_{mean} = 10 min. mean wind pressure

P_{rms} = rms of fluctuation component of wind pressure

q_H = 10 min. mean wind velocity pressure at the top of structure

Figure 5 (a) and (b) show the comparisons of mean and rms wind pressure coefficients between full-scale measurements and wind tunnel tests at center height of the walls.

Regarding to the mean wind pressure coefficients, the tendencies are similar to each other, but it is unquestionable that there are differences in value between full-scale measurements and wind tunnel tests. Table 2 shows the differences of mean wind pressure coefficient. These differences are mainly due to the reference pressure; internal pressure was used in full-scale measurement, which was supposed to be lower than atmospheric pressure. On the other hand, wind tunnel test used the static pressure in the tunnel as reference pressure, which seemed to correspond to the atmospheric pressure. Re-comparisons were carried out changing the reference pressure so as to minimize the differences. The results become those as shown in the 3rd column of Table 2. They are fairly small. But, they do not become zero with only taking into consideration of reference pressure. The other reasons are not found yet.

Regarding to the rms wind pressure coefficients, wind tunnel test results are almost 1/2 values of full-scale ones. Table 3 shows the ratios of rms wind pressure coefficient of full-scale measurements to wind tunnel tests. The ratios are in the range between 1.43 and 1.91 and similar to the ratio of turbulent intensity between full-scale measurements and wind tunnel tests. However, fluctuation components on side and leeward walls are said to have little influence of approaching flow. If it is true, the other reasons should be searched for the ratios on side and leeward walls.

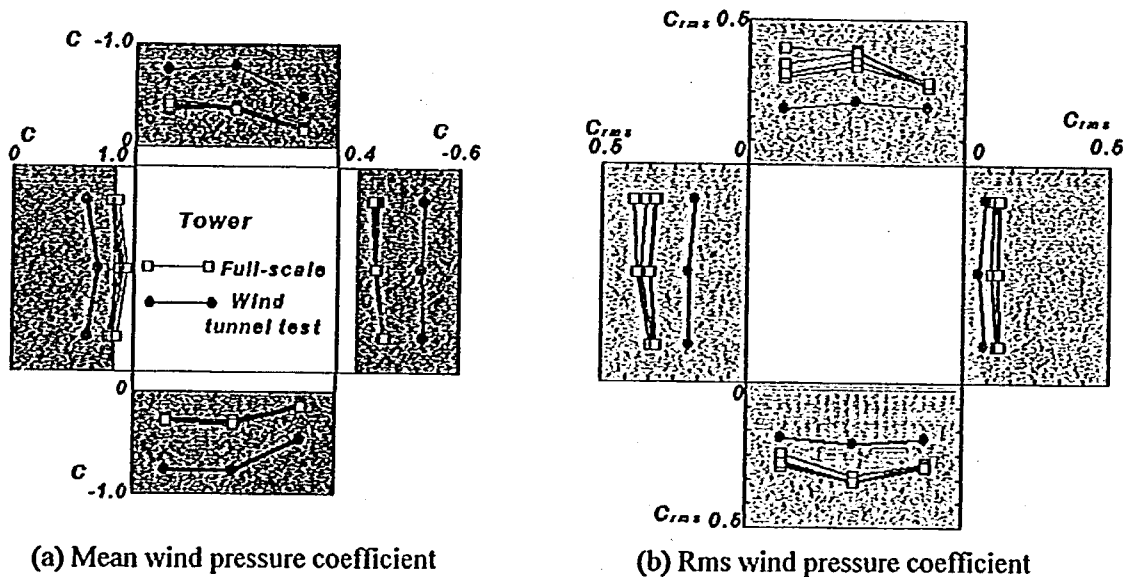


Figure 5 Comparison of wind pressure coefficient

Table 2 Differences in mean external wind pressure coefficient between full-scale measurement and wind tunnel test

Position	Difference	Difference minimized
windward surface	0.26, 0.24, 0.24	-0.11, -0.13, -0.13
east side surface	0.33, 0.47, 0.49	-0.04, 0.10, 0.12
west side surface	0.33, 0.40, 0.35	-0.04, 0.04, -0.02
leeward surface	0.48, 0.44, 0.39	0.11, 0.07, 0.02

Table 3 Ratios of rms wind pressure coefficient of full-scale measurement to wind tunnel test

Position	Ratio
windward surface	1.91, 1.79, 1.65
east side surface	1.46, 1.58, 1.44
west side surface	1.43, 1.74, 1.81
leeward surface	1.49, 1.88, 1.59

4.3 Power Spectra of fluctuation components of wind pressures

Figure 6 (a), (b), (c) show the comparisons of power spectra of fluctuation component of external wind pressures between full-scale measurements and winds tunnel tests. (a) is a spectrum on center of windward wall. (b) is the one on center of leeward wall. (c)'s are those on sidewall; c-1), c-2) and c-3) are windward side, center and leeward side of sidewall respectively.

Power spectra of full-scale measurement on windward wall and leeward wall are similar to each other. Furthermore they are similar to a spectrum of approaching wind velocity spectrum. But, looking at them in more detail, the power normalized of leeward is larger than the one of windward in higher frequency region. Comparing spectra of between wind tunnel tests and full-scale measurements, the peaks of wind tunnel tests are in the position at higher frequency than those of full-scale measurements. Regarding the degree of power decline with frequency increasing in the higher frequency region, spectra of wind tunnel tests are a little greater than those of full-scale measurements.

Power spectra on sidewall have two peaks. In the power spectra of the full-scale measurement, windward one shows the peak in lower frequency is higher than the one in higher frequency. But, leeward one shows the peaks' heights are almost same each other. The tendency on a change of peaks' height with moving to the leeward side is more remarkable in those of wind tunnel test; the relation on peaks' heights is opposite situation between windward and leeward.

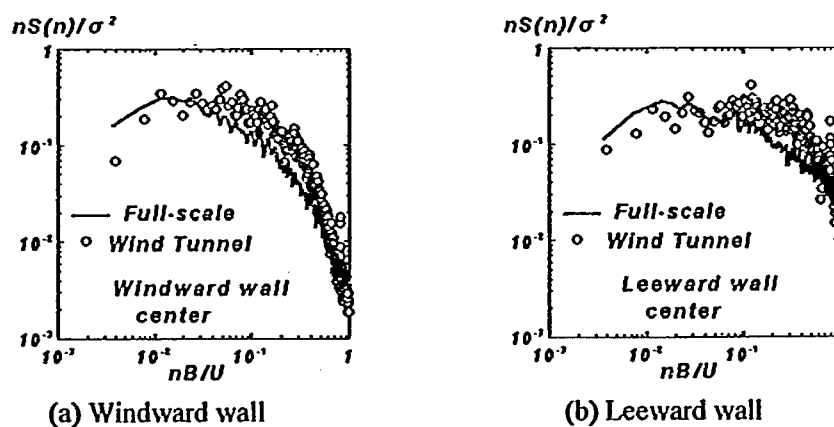


Figure 6 Power spectra of wind pressure components

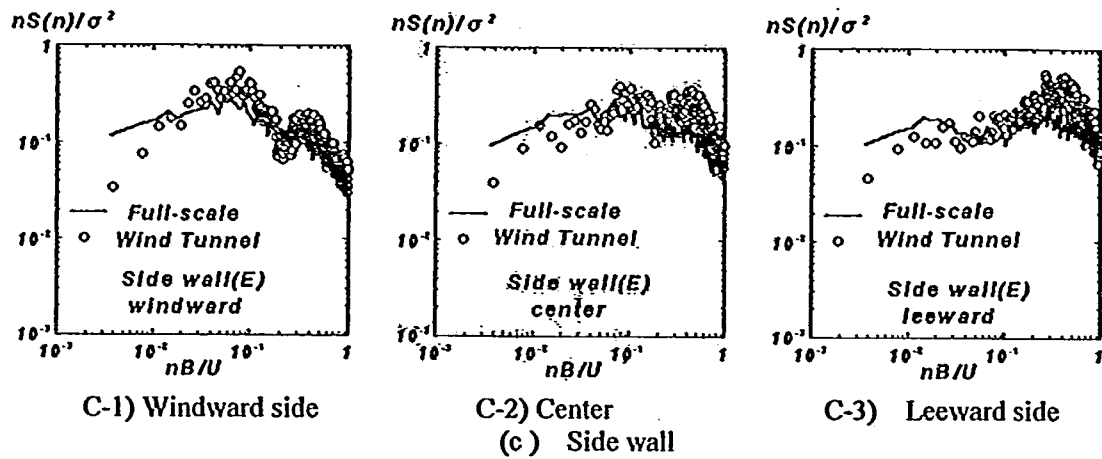


Figure 6 (continued)

5. Concluding remarks

Full-scale measurements and wind tunnel tests were carried out on pressure measurements on a tower-like structure.

The following results are obtained as the results of comparisons between full-scale measurements and wind tunnel tests.

Mean wind pressure coefficients obtained in the wind tunnel tests might fairly simulate to the ones of full-scale measurements by taking into consideration of reference pressure.

Rms wind pressure coefficients obtained in the wind tunnel tests are smaller than those in the full-scale measurements. In the application of the results of wind tunnel test to real designs, turbulent intensity of approaching wind should be taken into consideration, at least.

As regards power spectra of fluctuation component of wind pressure, wind tunnel test results show a little larger wave motion than those of full-scale measurement.

Appendix 1

Table List of Participated Members

Name	Belonging
Hisashi Okada :chairperson	Building Research Institute
Nobuyoshi Yamaguchi	ditto
Hitomitsu Kikitsu	ditto
Yoshihiko Hayashi	ditto
Yasunobu Ashie	ditto
Masaki Ohashi	ditto
Minoru Okabe	Center for Better Living, Tsukuba Building Test Lab.
Kenji Yasui	Technical Research Institute, Okumura Corporation
Taimei Aoki	Technical Research Institute, Hazama Corporation
Shinji Ito	Technical Research Institute, Konoike Corporation
Hiroshi Yokoyama	Technical Research Institute, Aoki Corp.
Tetsuo Matsuyama	Technical Research Institute, Kumagai Gumi Co., Ltd
Hiroshi Noda	Technical Research Institute, Mitsui Construction Co., Ltd.
Kiyomine Yoshikawa	Technical Research Institute, Tobishima Corporation
Morimasa Watakabe (secretariat)	Technical Research Institute, Toda Corporation

ROOF CORNER WIND FLOW AND PRESSURES

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ABSTRACT

Texas Tech University and Colorado State University have been working together for nearly ten years to study the effects of severe wind on low rise buildings as part of a Cooperative Program in Wind Engineering (CPWE) that is sponsored by the U.S. National Science Foundation. One of the four emphasis areas of research of the current CPWE is the characterization of the wind flow in the regions of flow separation above the roof. Full-scale synchronized data of wind-flow images, incident wind and roof suction pressures near the roof corner of Texas Tech Test Building have shown that a direct relationship exists between incident wind angle of attack, conical vortex structures, and the induced corner high suction pressures.

INTRODUCTION

Roof corners are critical regions of a building envelope where high suction pressures are generated by conical vortices (delta-wing vortices). To understand the mechanism of the cause of these high suction pressures, full-scale experiments at Texas Tech University (TTU) and wind-tunnel experiments at Colorado State University (CSU) have been pursued over the last several years. The goal is to establish a relationship between the upstream wind flow, the wind flow in the vicinity of the roof surface, and pressures in the roof corner region.

In an effort to clarify the conical vortex flow phenomenon in a wind tunnel, Banks et al. (1) at CSU have performed flow visualization studies using a laser light sheet; Kawai (2) has investigated the structure of conical vortex by hot-wire velocity measurements; and Marwood (3) has used laser doppler anemometry to measure the velocity field in a conical vortex.

Several flow-visualization techniques have been applied by Sarkar et al. (4) to visualize the separated-flow phenomena on the roof of the Texas Tech Test Building. Only those techniques that were suitable for full-scale observation have been used. These techniques include tuft-grid, airfoil-grid, and smoke-injection methods. While the airfoil-grid method captures a good view of the mean flow, the smoke-injection method is suitable for displaying the dynamic convection or three dimensionality of the flow. For the conical vortex case, the tuft-grid method was chosen because it could very effectively capture the characteristic size of the conical vortices in one plane.

EXPERIMENTAL SETUP AND METHODOLOGY

The schematic view of the experimental setup is shown in Figure 1. A 3-component sonic anemometer was selected to measure the incident wind speed. Vortex images were recorded by an 8-mm video camera at a speed of 30 frames/second. An LED (Light Emission Diode) was placed between the video camera and the tuft-grid to synchronize the video images with

the wind and pressure data that were simultaneously collected. Corner pressure taps were densely arranged close to the short edge of the roof. Figure 2 shows the layout of all the pressure taps in the test building roof corner and also defines the angle of attack (AOA), θ , ϕ , and x-y coordinate axes. Both wind and pressure data were collected at a sampling frequency of 30 Hz.

A 3-second averaging period method was chosen to study the characteristics of the conical vortex structure and the pressure distribution profile under the vortices (5). It was found that the overall structure of the vortices remained relatively stable during the specifically selected 3-second time period.

RESULTS AND DISCUSSION

Typical Conical Vortex Structure

Several typical conical vortex images are shown in Figure 3 to illustrate the difference of conical vortex structures as a result of different wind angles of attack. Figure 4 shows the variation of vortex angular position with different angles of attack. It is quite clear that a good linear relationship exists between ϕ , the horizontal vortex position, and θ (or AOA). It was also found that wind speed has negligible influence on the overall structure of conical vortex. Because of the small changes of vortex vertical position and the relatively poor tuft-grid resolution, the linear relationship between ϕ_h , the vertical vortex position, and θ is not as good as the others.

Pressure Distribution Profiles Underneath Vortices

The 3-second mean point pressures on line of taps $X=1.42$ m are determined. The highest and lowest pressure points are plotted in Figure 4 to compare with the vortex structures. It is observed that the vortex core almost coincides with the highest pressure point, whereas the flow reattachment point coincides with almost zero pressure point. This shows a direct relationship between vortex structure and the pressures underneath it: peak suction pressure point basically follows the corresponding vortex core position; the suction pressure drops sharply to nearly zero in the region close to and beyond the reattachment point.

Simple Model for Pressure Prediction

It has been observed that the influences of incident wind speed and the angle of attack can be separated to define the mean pressure (C_p) distribution at the roof corner. This hints the quasi-steady characteristic of roof corner pressures. A simple numerical model (6) was proposed to predict the instantaneous pressure coefficients from the mean pressure coefficient data and the incident wind data.

$$C_p(t_i) = C_p(\alpha_i) \times (U(t_i)/U)^2 \quad (1)$$

where $C_p(t_i)$ is the instantaneous pressure coefficient; $C_p(\alpha_i)$ is the curve-fitted mean pressure coefficient as a function of α_i (AOA); $U(t_i)$ is the measured incident wind speed, and U is the run-averaged wind speed.

Given the simplicity of Eq. (1), the comparison between the predicted and the measured pressures is satisfactory.

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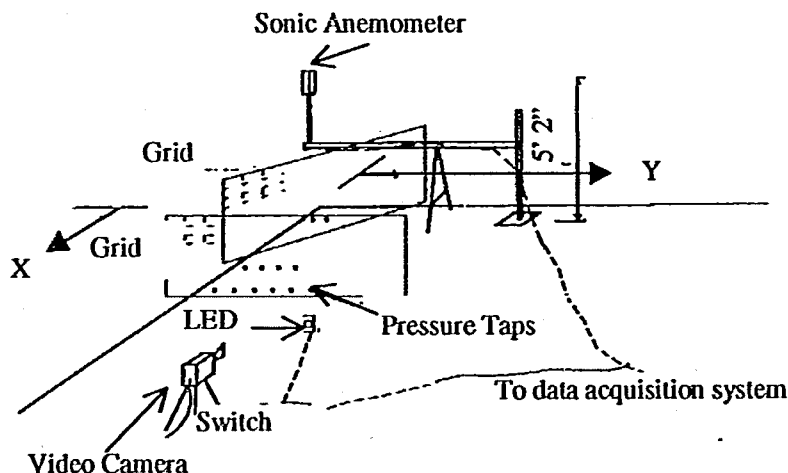


Figure 1. Schematic of synchronized video and wind-pressure data acquisition system

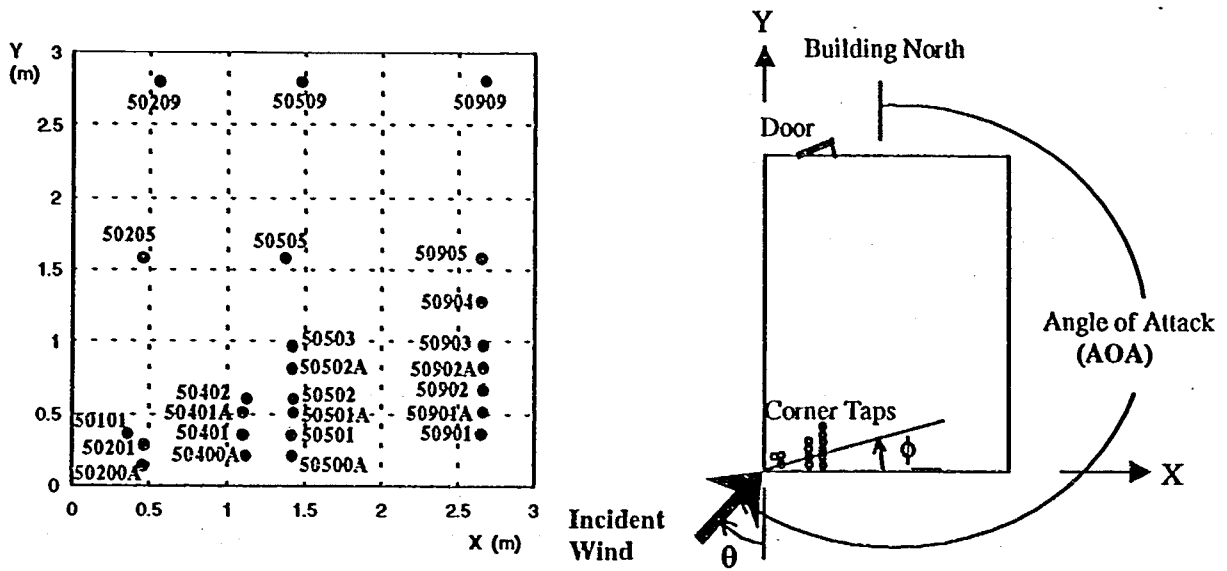


Figure 2. Corner pressure tap locations and definition of coordinates and angles



Figure 3. Typical vortex structures at $x=1.43$ m for different angles of attack

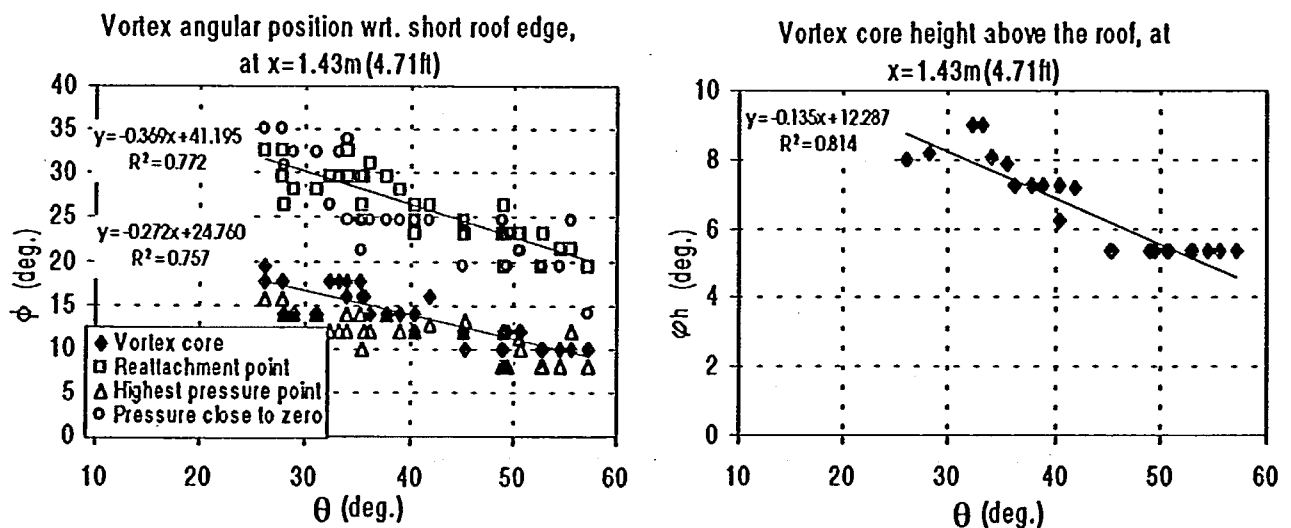


Figure 4. Conical vortex position for different angles of attack

Extreme Load Distributions on Buildings with a Rectangular Plan

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ABSTRACT

This paper first discusses the quasi-static wind load effect combination to be considered in structural design of low-rise buildings. Various load effect combinations, e.g. across-wind base shear and torsional base moment accompanied by maximum along-wind base shear, are analyzed and discussed. Then, actual wind pressure distributions causing maximum quasi-static load effects at the base and in the structural frames are conditionally sampled and examined.

INTRODUCTION

The necessity of capturing the maximum load effects was first introduced by Davenport (1961) as a gust factor. Solari (1989) attempted to develop Davenport's gust factor technique to more generic maximum load effects based upon the response spectrum technique. Regarding low-rise buildings, Holmes (1988) studied the distributions of instantaneous wind pressures along a gabled roof frame producing the peak loads and load effects on the frame of a low-rise building model. Considerable variation was found in these instantaneous pressure distributions. Tamura et al. (1992) discussed the maximum load effects on beams supporting flat roofs based on instantaneous wind pressures simultaneously measured at up to 512 points. The gust response factors for various load effects on rigid and elastic beams were examined and the influence of the turbulence intensity and the integral scale were demonstrated. The load-response-correlation

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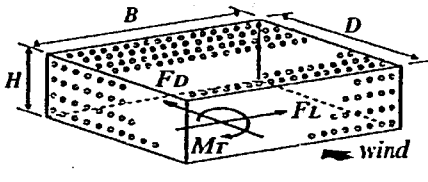
(LRC) method was proposed by Kasperski (1992) to take into account a realistic spatial distribution of wind loads that produce accurate peak structural responses. Holmes (1992) extended Kasperski's LRC approach to incorporate proper orthogonal decomposition (POD).

WIND TUNNEL DATA

Fluctuating wind pressure records obtained in a boundary layer wind tunnel 2.6m wide $\times$ 2.4m high were analyzed. Two building models with flat roofs were tested in the two different turbulent shear flows as shown in Table 1, where B and D are the dimensions of the building plan and H is the height. The wind direction was normal to the wall of length B . The geometrical scale of the turbulent shear flows was 1/250, and the power law index α of the mean wind speed profile was set at $\alpha = 1/4$ and $1/6$. Wind pressures at 416 and 512 locations uniformly distributed on the surfaces of Type 1 Model and Type 2 Model, respectively, were simultaneously sampled every 0.00128s. The total sample length was 62.91s (49152 data) for Type 1 Model and 41.94s (32768 data) for Type 2 Model. Tubing effects were numerically compensated by the gain and phase shift characteristics of the pressure measurement system.

Table 1 Model dimensions

Model	B (mm)	D (mm)	H (mm)
Type 1	170	120	50
Type 2	200	200	50



WIND LOAD COMBINATIONS ACTING ON LOW-RISE BUILDING MODELS

Quasi-static along-wind base shear, across-wind base shear and torsional base moment

The fluctuating pressures were integrated to obtain the quasi-static base shear F_D and F_L and the base moment M_T . They are expressed in non-dimensional forms based on the velocity pressure q_H at roof height: along-wind shear force coefficient $C_D = F_D/q_H B H$, across-wind shear force coefficient $C_L = F_L/q_H B H$, and the torsional moment coefficient $C_T = M_T/q_H B H R$, where $R = (B^2 + D^2)^{1/2}$.

Combinations of quasi-static load effects at the base

The wind tunnel records were divided into sub-runs whose lengths correspond to almost 10min in full-scale, and eighteen 10min runs and fifteen 10min runs were obtained for $\alpha = 1/4$ flow and $\alpha = 1/6$ flow, respectively, for each model. From each 10min run, the maximum value of one of the load effects, i.e. along-wind base shear, across-wind base shear or torsional base moment, was selected and the other two load effects simultaneously recorded were picked up. For example, across-wind base shear coefficient and torsional base moment coefficient are picked up at the moment when the maximum along-wind base shear coefficient C_{Dmax} was recorded, and they are indicated as $C_L(C_{Dmax})$ and $C_T(C_{Dmax})$. The other combinations $\{C_{Lmax}, C_D(C_{Lmax})$ and $C_T(C_{Lmax})\}$ and $\{C_{Tmax}, C_D(C_{Tmax})$ and $C_L(C_{Tmax})\}$ were also captured when the maximum across-wind base shear and the maximum torsional base moment, respectively, were recorded.

Load effect combinations

Figures 1(a) - 1(d) show the results for Type 1 Model when the maximum along-wind base shear C_{Dmax} was recorded for $\alpha = 1/4$ flow. Figure 1(a) shows the combination of maximum along-wind base shear coefficient C_{Dmax} and the simultaneously recorded across-wind base shear coefficient $C_L(C_{Dmax})$. The value of C_{Dmax} lies between 2 and 4, and $C_L(C_{Dmax})$ lies between 0 and 0.7. The abscissa of Fig.1(b) shows the ratio of the simultaneously-appearing across-wind base shear coefficient $C_L(C_{Dmax})$ to its maximum value C_{Lmax} for each 10min run. When the maximum along-wind shear occurs, 20% - 70% of the maximum across-wind shear were simultaneously recorded, and 20% - 30% appears most frequently. Figures 1(c) and 1(d) are for the torsional base moment coefficient $C_T(C_{Dmax})$ accompanied by the maximum along-wind base shear. The value of $C_T(C_{Dmax})$ lies between 0 and 0.6, and between 0% and 100% of the maximum torsional moments were simultaneously recorded with the maximum along-wind base shear.

Figures 2(a) and 2(b) show the cases where the maximum across-wind base shear was recorded. Only the ratios of the along-wind base shear coefficient $C_D(C_{Lmax})$ and torsional base moment coefficient $C_T(C_{Lmax})$ to their maximum value are shown. 20% - 80% of the maximum along-wind base shear and 0% - 60% of the maximum torsional moment are accompanied by the maximum across-wind base shear.

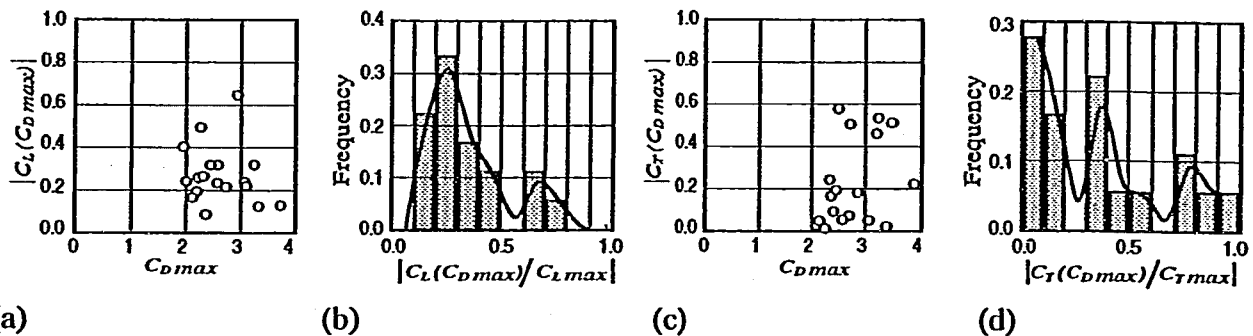


Fig.1 Across-wind base shear and torsional base moment accompanied by maximum along-wind base shear coefficient C_{Dmax} (Type 1 Model, $\alpha = 1/4$), (a) Across-wind base shear coefficient $C_L(C_{Dmax})$, (b) Frequency distribution of $C_L(C_{Dmax})/C_{Lmax}$, (c) Torsional base moment coefficient $C_T(C_{Dmax})$, (d) Frequency distribution of $C_T(C_{Dmax})/C_{Tmax}$

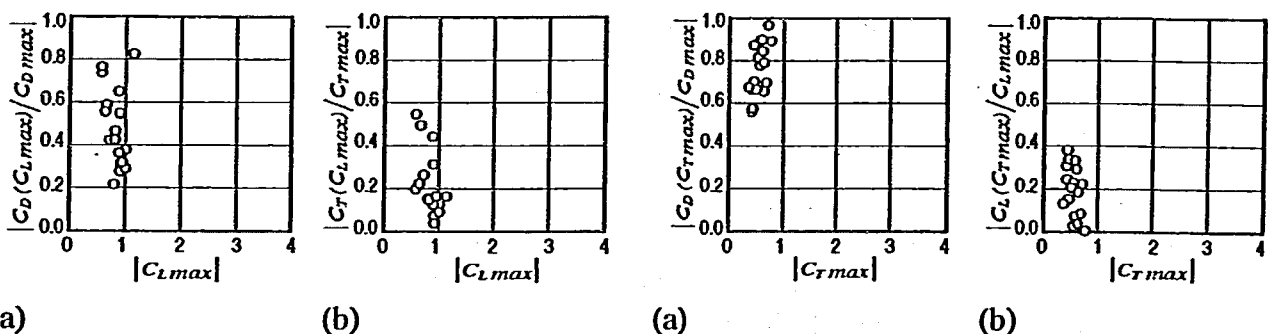


Fig.2 Along-wind base shear and torsional base moment accompanied by maximum across-wind base shear coefficient C_{Lmax} (Type 1 Model, $\alpha = 1/4$), (a) Along-wind base shear ratio $C_D(C_{Lmax})/C_{Dmax}$, (b) Torsional base moment ratio $C_T(C_{Lmax})/C_{Tmax}$

Fig.3 Along-wind and across-wind base shear accompanied by maximum torsional base moment coefficient C_{Tmax} (Type 1 Model, $\alpha = 1/4$), (a) Along-wind base shear ratio $C_D(C_{Tmax})/C_{Dmax}$, (b) Across-wind base shear ratio $C_L(C_{Tmax})/C_{Lmax}$

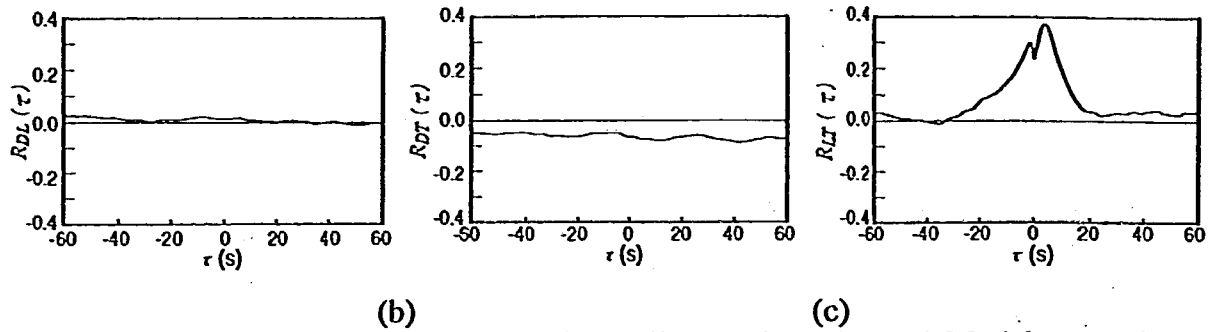


Fig. 4 Cross-correlation coefficients between load effects at base (Type 1 Model, $\alpha = 1/4$): (a) Along-wind and across-wind base shears; (b) Along-wind base shear and torsional base moment; (c) Across-wind base shear and torsional base moment

Figures 3(a) and 3(b) show the quite interesting results when the maximum torsional base moment C_{Tmax} was recorded. These figures clearly demonstrate the fact that almost the largest level of along-wind base shear jointly happens with the maximum torsional base moment, accompanying the lowest level of across-wind base shear. This fact suggests that the flow patterns causing the maximum along-wind base shear and the maximum torsional base moment might be almost the same, while the flow pattern causing the maximum across-wind base shear might be different.

Cross-correlation coefficients between wind load effects at the base

Figures 4(a) - 4(c) show examples of cross-correlation coefficients between the three components of wind load effects at the base for Type 1 Model and $\alpha = 1/4$ flow. No correlation is recognized between along-wind shear and across-wind shear, and only a very low correlation is observed between along-wind shear and torsional moment. The former coincides with the aforementioned results, demonstrating a low correlation between the maximum values of along-wind shear and across-wind shear. However, the latter does not coincide with the results suggesting a high-correlation between the maximum values of torsional moment and along-wind shear seen in Fig.3(a). However, the across-wind shear and torsional moment have some correlation, as shown in Fig.4(c), although their maximum values do not correlate, as seen in Fig.3(b). These contradictory relations between the average correlation of the time series and the correlation of the maximum values are very interesting.

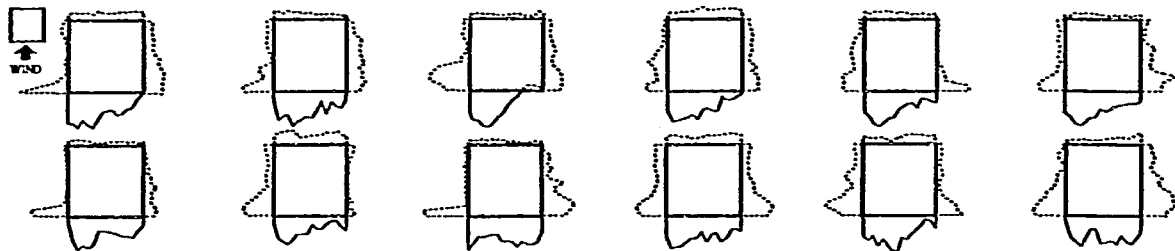
INSTANTANEOUS WIND PRESSURE DISTRIBUTIONS CAUSING MAXIMUM QUASI-STATIC LOAD EFFECTS

Quasi-static base shear and torsional base moment

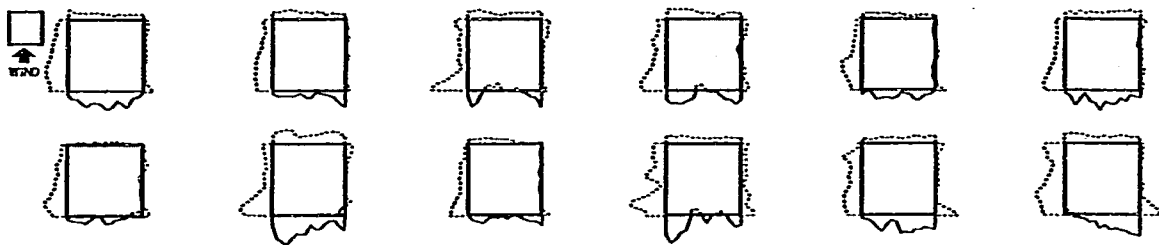
To examine the exact wind pressure pattern causing the quasi-static load effects, twelve runs and nine runs corresponding to almost 10min in full-scale were obtained for $\alpha = 1/4$ and $\alpha = 1/6$ flows, respectively, for Type 2 Model.

Figure 5(a) shows the instantaneous wind pressure distributions when the maximum along-wind base shear was recorded for twelve 10min runs for $\alpha = 1/4$ flow. Here, the pressure distributions are for the top layer (fourth layer) of the pressure tap arrangement on the model surfaces (see Table 1). There was no significant difference between the pressure patterns on the four layers. A typical pressure pattern is recognized among the twelve cases when the maximum

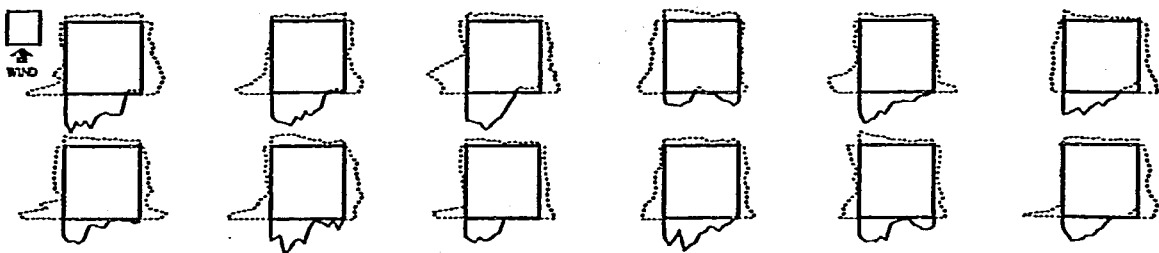
along-wind base shear was recorded. The positive pressure on the windward wall is high and clusters on one side. These asymmetric pressure patterns are not due to the inclination of the wind tunnel flow nor to any model irregularity. For ease of comparison, the right and left of some figures are turned. An extremely local high suction tends to appear near the leading edge of a side wall. This implies that the maximum drag is generated at an instance when the wind direction is slightly inclined to one side. Figure 5(b) shows the cases when the maximum across-wind base shear was recorded. The negative pressure on one side wall is large, while that on the other side is nearly zero. Figure 5(c) shows the cases where the maximum torsional base moment was recorded. Here, the pressure distributions are similar to the cases for the maximum along-wind base shear shown in Fig. 5(a). This result coincides with the fact that the largest level of the along-wind base shear is accompanied by the maximum torsional moment, as seen in Fig.3(a).



(a) When the maximum along-wind base shear was recorded



(b) When the maximum across-wind base shear was recorded



(c) When the maximum torsional moment was recorded

Fig. 5 Instantaneous wind pressure distributions causing maximum load effects at the base (Type 2 Model, Fourth layer of the pressure tap arrangement, $\alpha = 1/4$)

Bending moment and shear force in structural frames

The quasi-static bending moment and shear force were analyzed at various design points in Frames A and B for Type 2 Model. The plane of Frame A is perpendicular to the wind direction and that of Frame B is normal to wind. Three different supporting conditions at the column bases, i.e. pin - roller, pin - pin and fixed - fixed, were examined. Frames A and B are set at the

center row and the end row of the pressure tap arrangement, and the fluctuating wind loads acting on the frames were directly applied at the pressure tap positions based on the wind tunnel pressure data. The instantaneous pressure distributions causing maximum quasi-static load effects at various design points were examined. The extreme pressure patterns vary due to the design points and the load effects, as Holmes (1988) pointed out, and they are not necessarily the same as the mean pressure distribution.

Figures 6(a), 6(b) and 6(c) show examples of ensemble averaged pressure patterns causing maximum load effects. The mean pressure distributions multiplied by the ensemble averaged gust factor for each load effect are also shown in the figures. It is obvious that the mean pressure distributions are not necessarily appropriate for design wind loads.

The pressure patterns vary especially on the windward wall, depending upon the column base support conditions. They also vary depending upon the flow conditions.

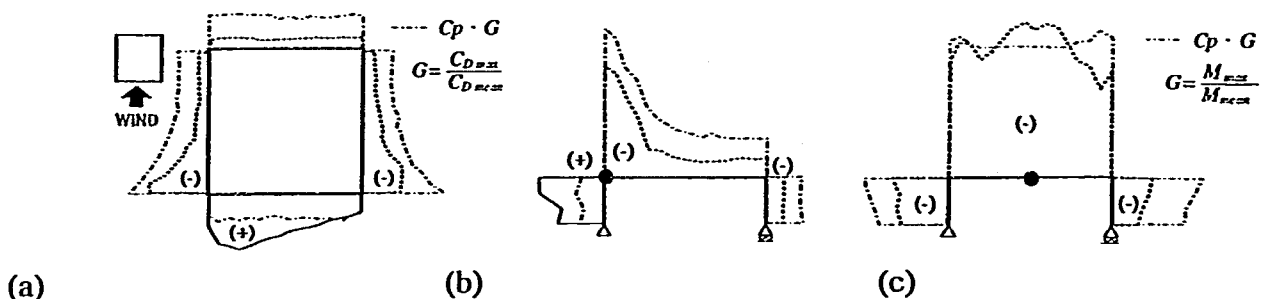


Fig.6 Ensemble averaged instantaneous pressure distributions causing maximum load effects (Type 2 Model, $\alpha = 1/4$): (a) Maximum along-wind base shear; (b) Maximum roof beam bending moment at windward end (Frame A, Pin-roller); (c) Maximum roof beam bending moment at center (Frame B, Pin-roller)

CONCLUDING REMARKS

Various load effect combinations have been examined on the basis of multi-channel wind tunnel pressure data, and the actual wind pressure distributions causing the maximum quasi-static load effects are conditionally sampled and examined. The results show the limitation of applying the mean pressure distribution as the design wind load.

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PERFORMANCE BASED OBJECTIVE DESIGN AND NATURAL DISASTER RISK MANAGEMENT

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Abstract

Improvements in the available means to communicate engineering information within and outside the engineering community, combined with the need for new procedures to manage losses associated with natural hazards, has started an evolution towards performance-based building design. The U.S. building code process is developing a performance based building code and has made initial efforts in developing performance based objective (PBO) design criteria for different loads. While PBO design criteria for wind loads have been used in project specific cases, information that can be used in establishing regulatory criteria does not exist. Yet the data and computational tools for developing PBO wind criteria are readily available. The increase in desktop computational capabilities have allowed designers to develop options that were not possible in previous years. Thus, design criteria can be developed to present performance options in a more consistent manner, especially when coupled with the collateral impact of natural disasters on other large scale interests (e.g., societal, national and local governments, finance and insurance industries, etc.). This paper presents one proposal for performance based objective design criteria for wind hazard.

PERFORMANCE BASED BUILDING CODES

The U.S. is currently developing a performance based building code under the auspices of the International Code Council (ICC). Deliberations began in 1997 and considered the Australian and New Zealand performance codes as a starting point. The New Zealand code was particularly concise in that it defines building systems and their required performance without any prescriptive criteria. For example, it requires structural systems to resist the forces imposed by dead and live loads and natural hazards. All of the requirements for determining the loads and resistances are delegated to reference documents that are also published and maintained by the New Zealand government. Calculation of wind loads and the mechanical properties of construction materials are contained in the reference documents.

The ICC Performance Code committee has proposed a variant of the New Zealand code; the performance of a facility is described abstractly, but different hazard and performance levels are provided for guidance in selecting a performance objective. The basic structure of Performance Based Objective (PBO) design criteria is the definition of hazard levels and performance levels which, when combined, provide a performance objective (Fig. 1)

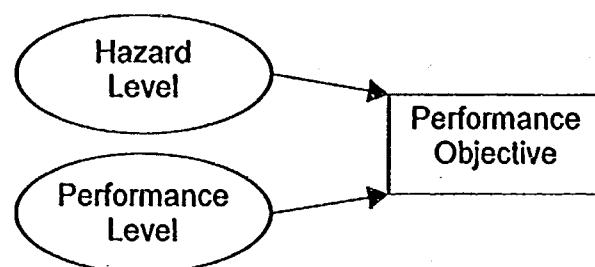


Figure 1. Definition of Performance Objective

Existing codes in the U.S. use single hazard and performance levels for the design of new structures (Table 1), although resource documents exist for seismic related criteria (viz., Vision 2000 (OES, 1995) for new structures and FEMA 273 and 274 for the rehabilitation of existing structures). A minimum hazard and performance level are prescribed in the performance code; however, the addition of other hazard and performance levels allow the designer to work with the owner—and other participants in the building process—to create structures more resistant to natural hazards in a consistent and measurable manner. While the hazardous events have been (or can be) defined mathematically (Table 2), the method of defining the wind environment associated with the hazard has not been established. Similarly, the different performance levels are described linguistically (Table 3), but means to measure the performance need to be identified. The ICC Performance Code is a significant improvement over existing U.S. prescriptive codes, although the committee has much work yet to do.

Table 1. Single Performance Objective Design Criteria

Hazard	Exceedance Probability (Probability in t years)	Return Period	Performance Level (Expected Damage)
Seismic	0.10 in 50	475	Life Safety
Flood	0.39 in 50	100	Minimal/ Moderate
Wind	0.63 in 50	50	No/Minimal

Table 2. Multiple Hazard Levels Defined in ICC Draft

Wind Design Hazard	Average Return Period	Probability of Exceedance
Small	25 years	32% in 10 years
Medium	50 years	63% in 50 years
Large	100	39% in 50 years
Very Large	200 years	22% in 50 years

Table 3. Example of Multiple Performance Levels

Traditional Structural Concerns	Performance Level Definition
Serviceability	Fully Operational
Damage Mitigation	Operational
Damage Without Collapse	Life Safety
Collapse Limit State	Near Collapse (Collapse Prevention)

The intent of the performance code is to provide the developers and design professionals greater flexibility and innovation in design, although the current prescriptive¹ criteria are deemed to have satisfied the proposed performance criteria. Additionally, the committee needs to determine if the resource documents will be established and maintained by the ICC or whether non-ICC documents will be adopted by reference.

COLLABORATIVE EFFORTS

Traditional codes and design guides express criteria for a single performance objective, but the complex and tedious design procedures used for a single performance objective do not lend themselves to easy adaptation to multiple performance objectives. Thus, both hazard analysis and performance criteria are areas of research that can be explored in conjunction with the new performance codes. Additionally, the use of agents should allow the remote collection of data for the determination of wind environment associated with a hazard level.

Hazard Analysis Methodology

A hazard analysis will need to be part of the new design method or maps for different regions can be made based upon historical data. With the current ability and capacity of desk-top computers, a site specific hazard analysis methodology should be able to be developed that would satisfy the multi-level performance criteria.

Resistances

The method for establishing resistances needs to account for the multiple levels of performance given in the PBO design process. Research will need to be carried out to codify acceptable serviceability and failure criteria. Specifically, for each construction material, means to quantify the following:

- Damage criteria
- Component and system failure
- What constitutes satisfying a performance level.

These items are not specifically addressed in current building criteria, as single performance objective criteria are satisfied by providing sufficient resistance according to a material specification (e.g., ACI-318, AISC, etc.); damage and failure are not part of the current decision making process. However, some performance levels in PBO design permit the occurrence of damage, and even failure of components. Therefore, methods need to be established to measure damage and damage states need to be defined so that performance levels in PBO design have crisply defined targets.

Use of Remote Data

Hazard analyses are currently performed using hazard maps for a specified return period (probability of exceedance). The introduction of multiple levels of hazard requires

¹ The term "prescriptive code" has two meanings: 1) any criteria not written in performance language (including current building codes); 2) current prescriptive criteria (e.g., the One and Two Family dwelling code).

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Feasibility of CFD Technique for Prediction of Wind Flows and Wind Loading

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ABSTRACT

More than ten years have passed since we investigated a potential of the technique of computational fluid dynamics(CFD) in wind engineering. It is found that there exist a few problems solved by us, in order to realize the practical use of CFD technique. One of importance is that the object treated by us is always in the turbulent flow. On the basis of the numerical results for wind related problems of buildings and structures, the feasibility of prediction of the following values for structural design is discussed. 1) aerodynamic forces and pressures on a building in turbulent boundary layer. 2) wind speed affected by complex terrain.

Especially we focus on the turbulence effects on the aerodynamics of a building. Recent development of computational fluid dynamics(CFD) technique gradually realize the numerical simulation of wind flows around a building placed on the ground surface in the turbulent boundary layer.

INTRODUCTION

Turbulence is essential to wind engineering, because wind flow is unsteady with various scaled eddies and the effect of turbulence characteristics on aerodynamic behavior of a bluff structure is very large. Accordingly we have to investigate details of interaction between turbulence and the separated shear flows around a bluff body. However with regard to the current status of computational fluid dynamics, almost problems are treated under the condition in the uniform flow. For estimation of wind flows and wind loading acting on a tall building or a space structure with long span, thus far the wind tunnel technique has been mainly employed. Therefore, at the previous UJNR meeting, I focus on the aerodynamic and aeroelastic problems of a cylinder with various kinds of sectional shapes(Tamura, 1997). On the basis of the physical mechanism of drastic phenomena due to corner modifications, the realization of aerodynamic control is examined. Also under various conditions for free oscillations of a rectangular and a circular cylinders, aeroelastic behaviors are investigated in detail.

On the other hand, recent development of computational facility and numerical algorithm based on the parallel computers realizes to solve above problems gradually as well as partly. First this paper brings into focus the aerodynamic forces and pressures on a building in turbulent boundary layer. For a numerical model, DNS(Direct Numerical Simulation) and LES(Large Eddy Simulation) are employed. Next by using turbulence modeling or the higher-order upwind scheme, I numerically examine the wind flow characteristics over various types of terrain. Finally the scope and the current status for the practical use of CFD technique in wind engineering are discussed.

DIRECT NUMERICAL SIMULATION OF WIND FLOWS AROUND A BUILDING WITH ASPECT RATIO=2

First example(Nozu and Tamura, 1999) is the DNS of the flow around a 3D building in

turbulent boundary layer, whose computational model is shown in Fig.1. There are two regions, one is for the generation of the turbulent shear flows and the other is a model for a building on the ground surface. The Reynolds number is about 1800. Fig. 2 displays two types of vertical profile of wind velocity obtained by using roughness block or not. Instantaneous pressure contours are exhibited in Fig. 3. In the case of the turbulent boundary layer on smooth surface, we can recognize the turbulent structures with the various scaled eddies. According to streamlines in the horizontal section at $z/D=1.0$ in Fig. 4, in case of turbulent flow, the wake of a body becomes narrow and the formation location of twin vortices obtained by time-averaging moves leeward. So CD value is larger and the Strouhal number is higher. Fig. 5 shows the pressure distribution around a building. The averaged pressure coefficients in the case of turbulent boundary layer decrease the magnitude on the frontal and back surfaces. We can recognize constant value of the pressure fluctuation on the frontal surface, which is generated by the turbulent characteristics in the oncoming flow. Fig. 6 illustrates the streamlines in the central vertical section. In case of turbulent flow, the leeward vortex becomes weak and goes apart from a body. This flow structure is consistent with the averaged pressure coefficients in the central vertical section in Fig. 7.

LARGE EDDY SIMULATION OF WIND FLOWS AROUND A LOW-RISE BUILDING

Second example(Nozawa and Tamura, 1999) is the LES of the flow around a low-rise building in turbulent boundary layer. The Reynolds number is equal to 28700, which is higher than the first example. So we have to use the finer mesh system for the region close to the ground surface. Fig. 8 exhibits the computational model of domain decomposition technique. In Fig. 9 the vertical profile of oncoming wind flow is set to $1/7$ shear flow. Fig. 10 shows the pressure distribution in the vertical section around a low-rise building. In case of turbulent flow, the averaged pressure coefficients recover in the leeward region of the roof. According to the vorticity contours in the vertical section in Fig. 11, we can recognize the reattachment of the separated shear layer on the leeward of the roof in case of the turbulent boundary layer.

TERRAIN EFFECTS ON WIND FLOWS

In case of strong wind over relatively simple shape of terrain such as a 2D escarpment or hill, the computational velocity profiles are compared with the previous experimental ones. Without separation, concerning turbulent flows over various terrains, the computational results are in good agreement with the experimental ones(Fig. 12). As another example of weak wind, We show the thermally stratified wind flows. We carry out the numerical simulation of three-dimensional flow around a hill with various kinds of stratification(Fig. 13). We can simulate the different flow pattern according to the stratification. Next the wind flows around the complex terrain are computed as shown in Fig. 14. We can simulate the wind flow which concentrates along the valley and the wake patterns behind a small island.

CONCLUSIONS

The current status of CFD(computational Fluid Dynamics) technique for prediction of the wind loading on buildings and structures is shown. It can be concluded that the recent development of the numerical simulation for the turbulent flows are remarkable. Also, next generation of the supercomputers is very much expecting due to the parallelization. So we can say that we are

now standing at the site immediately before the realization of the practical use of CFD technique for the prediction of the wind flows and the wind loading.

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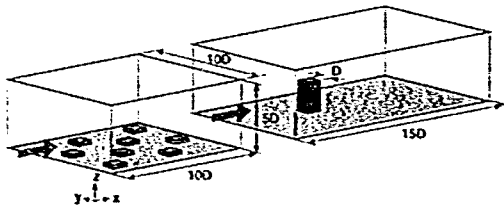


Fig. 1 Computational model for 3D building in turbulent boundary layer

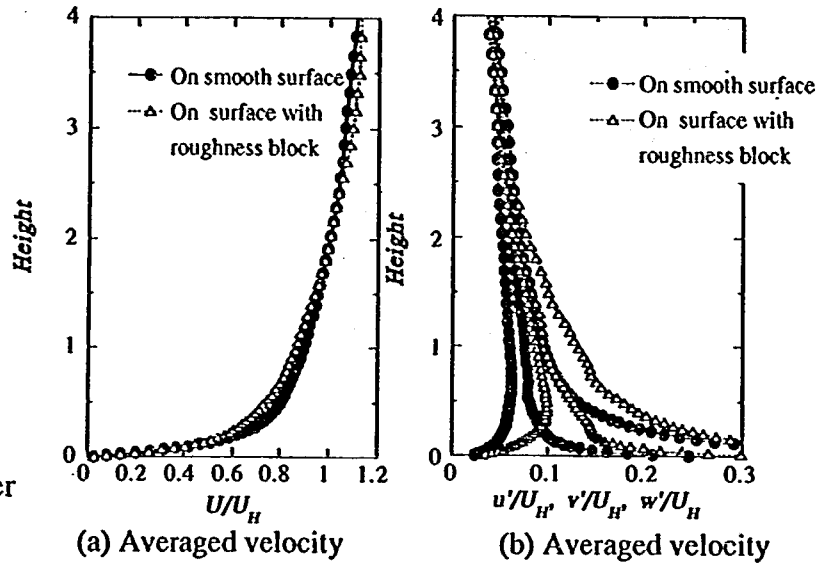


Fig. 2 Vertical profile of wind velocity

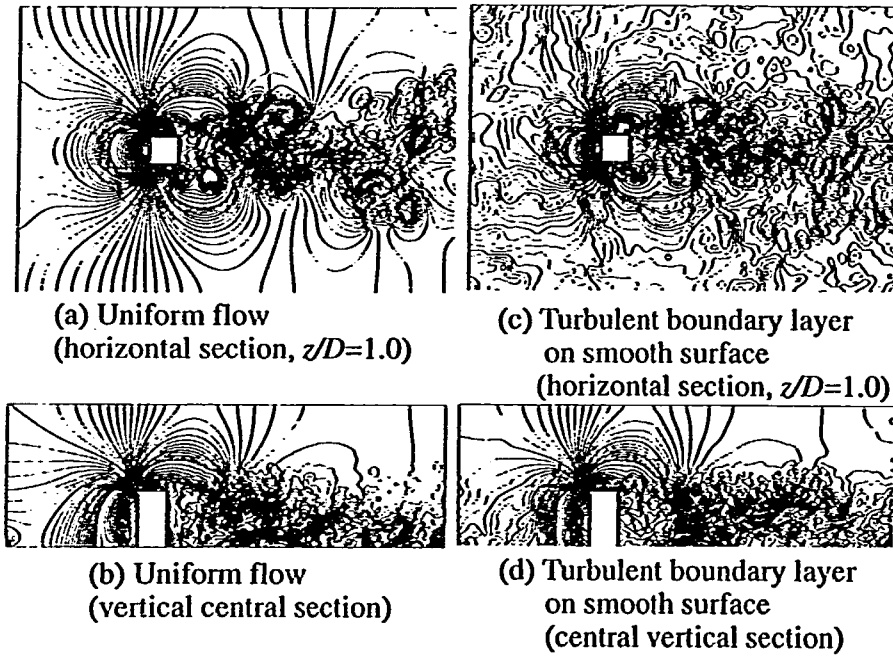


Fig. 3 Instantaneous pressure contour

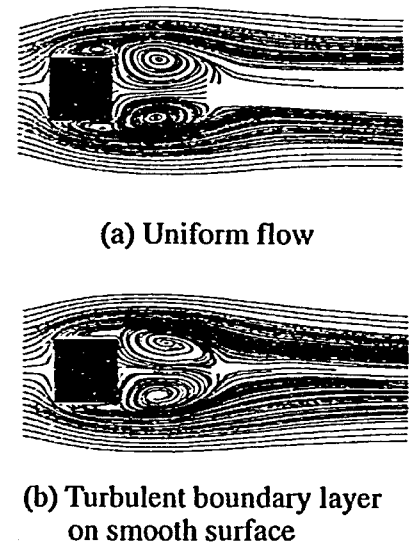


Fig. 4 Streamlines in the horizontal section at $z/D=1.0$

	Computation			Wind tunnel test		
	Uniform flow	Turbulent boundary layer		Uniform flow	Turbulent boundary layer	
		On smooth surface	On surface with roughness block		$\alpha = 1/8$	$\alpha = 1/4$
C_D	1.23	0.98	0.91	1.25	1.00	0.98
Strouhal number	0.113	0.099	0.096	0.095	0.086	0.090

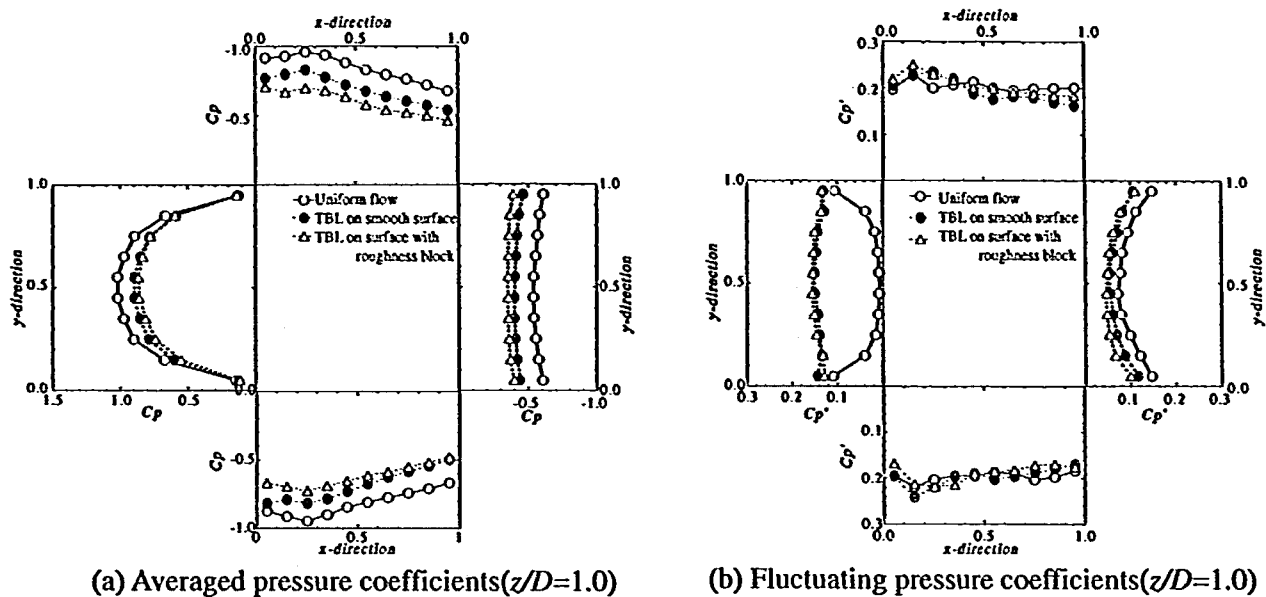


Fig. 5 Pressure distribution around a building

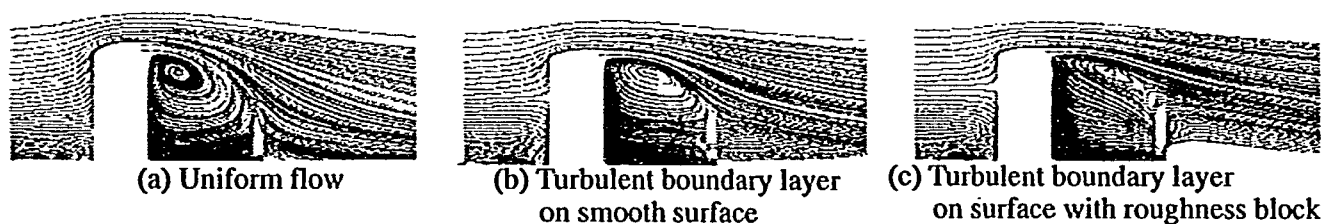


Fig. 6 Streamlines in the central vertical section

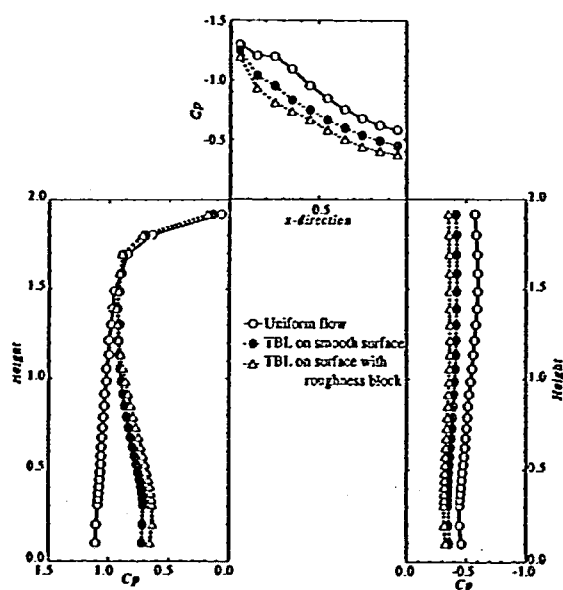


Fig. 7 Averaged pressure coefficients in the central vertical section

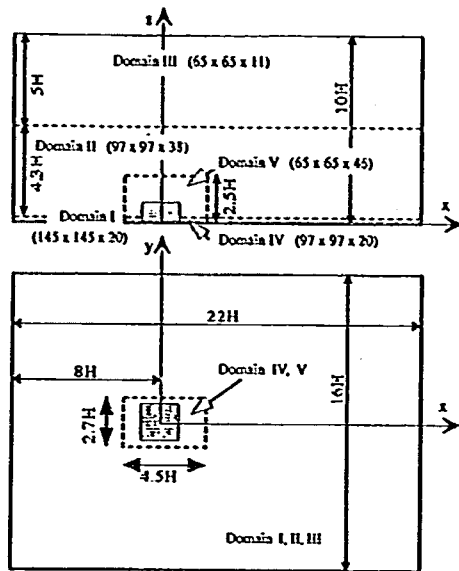


Fig. 8 Computational model of domain decomposition technique

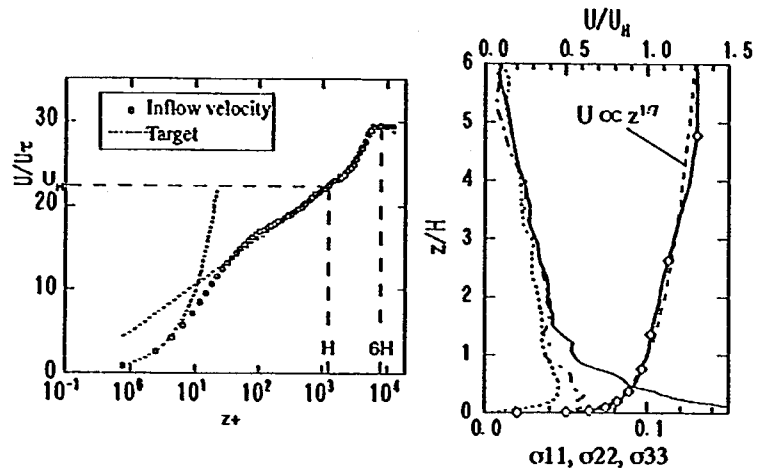
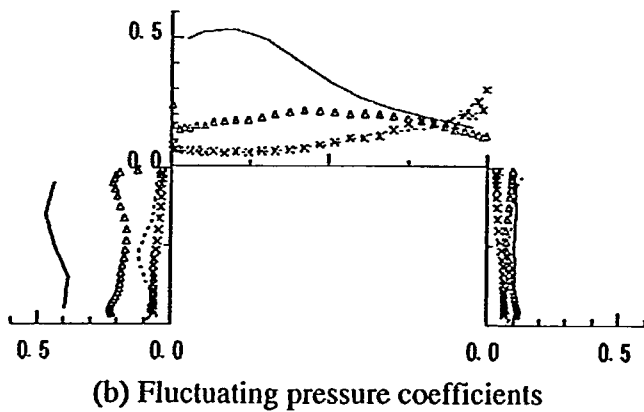
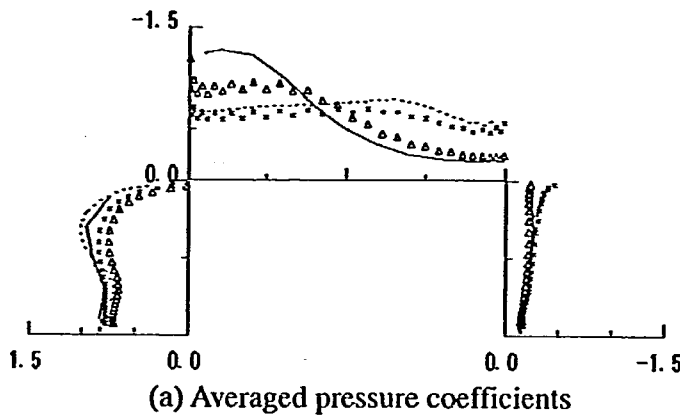


Fig. 9 Vertical profile of oncoming wind flow



- Δ Turbulent boundary layer(LES)
- $\times$ Laminar boundary layer(LES)
- Solid line : 1/4 shear layer(Exp.)
- Dashed line : Uniform flow(LES)

Fig. 10 Pressure distribution in the vertical section around a low-rise building

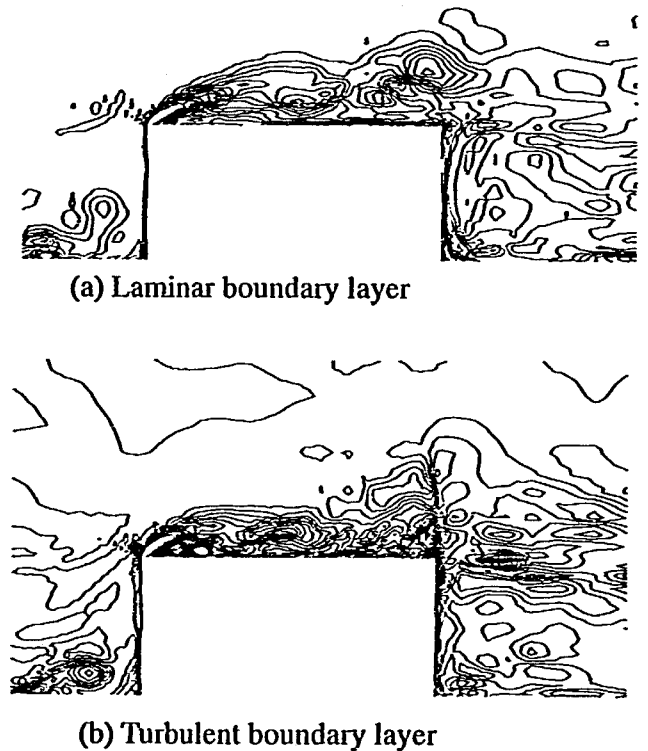
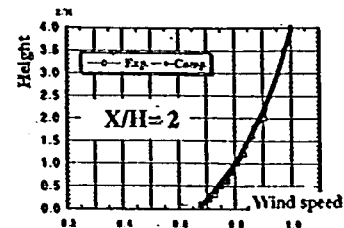
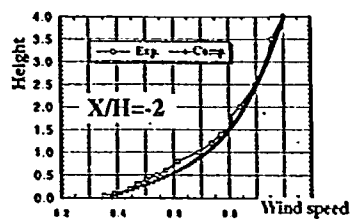
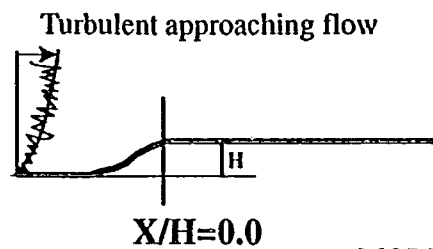
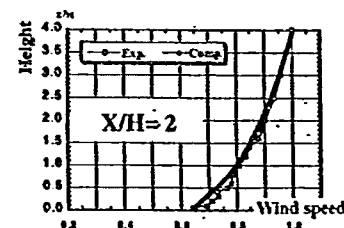
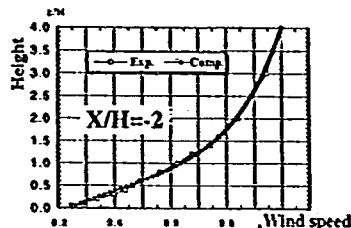
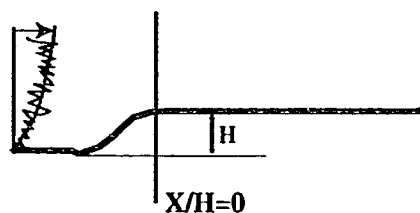


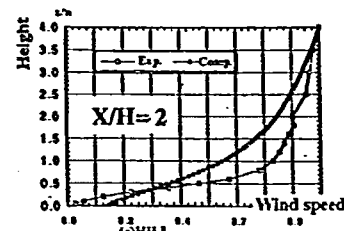
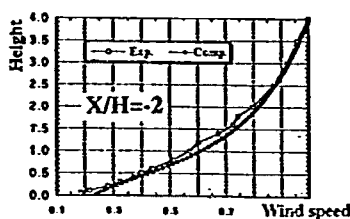
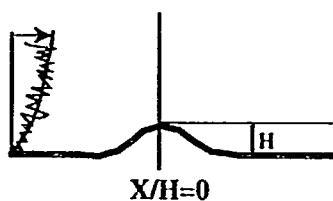
Fig. 11 Vorticity contours in the vertical section around a low-rise building



MODEL A Escarpment(Maximum slope: 30 deg)



MODEL B Escarpment(Maximum slope: 45 deg)

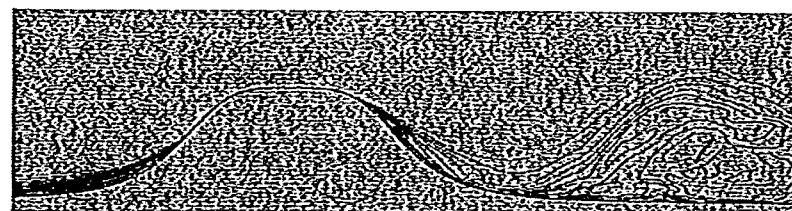


MODEL C Hill(Maximum slope: 45 deg)

Fig. 12 Terrain effects on turbulent wind flow profile
-Comparison between experiments and computations



(a) Neutral flow



(b) Thermally stratified flow

Fig. 13 The difference of flow structures between neutral and thermally stratified flows



(a) The computational model



(b) Patterns of surface streamlines on the terrain

Fig. 14 The computation of three-dimensional flow around the complex terrain

TOWARDS REFINED MODELING OF WIND EFFECTS THROUGH COLLABORATIVE RESEARCH

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ABSTRACT

A number of general observations on wind engineering, its growth and recognition in the U.S. and Japan, as well as motivation and possible benefits of coordinated US-Japan collaboration in wind engineering are presented for discussion. Two research topics are proposed for collaborative research. They are primarily related to refined laboratory modeling of (1) wind loading on low-rise buildings and (2) aerodynamic performance of long-span bridges. It is suggested that potential benefits and visibility of a coordinated US-Japan collaboration in wind engineering are likely to improve societal recognition of this area in both the countries.

INTRODUCTION

Wind engineering is a young discipline which only recently evolved into an independent field of civil engineering. Wind engineering community remains relatively small. This is apparent when this group is compared with other branches of civil engineering, e.g. earthquake engineering. In addition, societal appreciation of benefits resulting from (past and current) research and practice-oriented activities in wind engineering appears to be significantly lower than desired. It seems that the undervalued perception of wind engineering by society at large may not change in the near future, unless an effective strategy to overcome the present status quo is developed and vigorously implemented. The foregoing conclusion is in major part based on the experience in the U.S., where a lively debate addressing this issue has been underway for some time. It would be interesting to compare in the above context the current status of wind engineering in the U.S. and in Japan. It is anticipated that this Workshop will provide opportunities to exchange opinions on this topic.

It has been recognized that achieving a "critical mass" level is one of prerequisites for an accelerated growth and a wider recognition of any professional group, organization, etc. One of ways to accomplish this in research is through collaborative activities involving members of various teams. Modern research in many fields (especially in engineering) frequently relies on and is conducted through collaboration among individual researchers and/or research groups. It seems that a collaborative research effort has better chances for a wider recognition, support and more stable funding, when it is carried under a unifying umbrella, which gives its members recognizable identity to the outside world. An example of such an arrangement, implemented in wind engineering in the U.S., is the CSU/TTU Collaborative Program in Wind Engineering (CPWE), established and funded by the U.S. National Science Foundation. It involves faculty, staff, and students (graduate and undergraduate) from various departments of two universities: Colorado State University (CSU) and Texas Tech University (TTU). The structure and early activities of the CPWE have been presented to the UJNR audience during the 27th Joint Meeting of the UJNR Panel on Wind and Seismic Effects (Bienkiewicz, 1995). Establishment of a US-Japan counterpart of the CPWE, labeled as, say, the US-

Japan Cooperative Program in Wind Engineering (UJCPWE) might lead to a wider recognition of benefits of wind engineering by general public and funding agencies, both in the U.S. and in Japan. The UJCPWE could function under auspices of UJNR and be coordinated (for example) by the Technical Committee E. It is proposed that the idea of establishing the UJCPWE be discussed during the Workshop.

This paper first addresses the need for and possible benefits of collaborative US-Japan activities in wind engineering. An attempt is made to present this topic in a broader, nontechnical context. Next, two candidate topics for collaborative US-Japan wind engineering research are proposed: (1) Assessment/Development of Refined Techniques for Modeling of Wind Loading on Low-Rise Buildings, and (2) Case Study of Long-Span Bridge Aerodynamics. The first topic can be viewed as an extension of the research effort carried out through the CPWE. Progress on this effort is briefly summarized in the paper. The second topic, appears to be of interest to the U.S. Federal Highway Administration.

MOTIVATION FOR US-JAPAN COLLABORATION IN WIND ENGINEERING

Recognition of societal impact of severe winds has been growing in recent years, both in industrialized countries, such as Japan and U.S., as well as in developing countries. This realization has primarily stemmed from overall statistics showing increasing economic losses and continued human suffering caused by high wind events. In the U.S., a high level of hurricane/wind activity is forecasted for the near future. At the same time, it is expected that migration of U.S. population towards coastal and other wind-exposed areas will continue. These facts point towards a potential for an increase, over the next decades, in societal vulnerability to wind hazard in the U.S.

The importance of wind hazard has been also recognized in Japan, where high winds associated with typhoons and other severe wind phenomena have significantly affected design of many civil work projects. Design and construction of long-span bridges are examples of civil engineering activities significantly affected by wind considerations. The importance of issues of wind hazard appears to be further heightened in Japan by increasing number of designed and constructed tall buildings and other wind-sensitive structures, as well as continued urbanization in coastal and other wind-exposed areas.

Both in the U.S. and in Japan, national issues associated with wind hazard appear to be of main concern. However, there has been a growing recognition in both the countries, that wind hazard should be also viewed in an international context, with the highest priority assigned to countries in neighboring regions: Latin America in case of the U.S. and South-East Asia in case of Japan. A number of arguments can be listed to support this broadened view of the wind hazard mitigation problem. They range from humanitarian aspects through technology transfer in wind engineering and other disciplines. Potential benefits of such an approach include also enhanced commerce, business and employment opportunities, as well as anticipated favorable socio-political repercussions.

U.S. and Japanese wind engineering researchers and practitioners are considered to be in forefront of this discipline. Differences in approach to research and practice in engineering, resulting from tradition, availability of natural resources, and socio-geo-political facets, led to somewhat different development of wind engineering in these countries. However, this diversity appears to be of complementary nature, with a great potential for creating "added value" in long-term cooperative research and technology transfer in this area.

PROPOSED TOPICS FOR COLLABORATIVE WIND ENGINEERING RESEARCH

Assessment/Development of Refined Modeling of Wind Loading on Low-Rise Buildings

Wind loading provisions of many codes and standards are based on the laboratory data resulting from wind tunnel studies using small building models. It has been recognized that the "first generation" laboratory results, used in some of the codes, could be improved, thus leading to more reliable foundation for updates of the code wind specifications. One of the objectives of the CPWE has been to evaluate the existing wind-modeling techniques and, if warranted, to develop refined techniques, tailored to studies of wind loading on low-rise buildings, using models of reasonable (not too small) size.

Over the years, a couple of tasks of the CSU component of the CPWE addressed this problem through laboratory investigations. Testing typically consisted of the following two phases: (1) Development of wind-tunnel arrangement to generate turbulent boundary-layer wind flow representative of typical wind conditions at TTU field site (geometrical scales ranging from 1:100 through 1:25), and (2) Measurement of wind-induced pressure on models of the TTU test building. A number of the laboratory arrangements, capable of reproducing the desired TTU wind conditions were developed and employed in measurement of the building pressure. Overall, the laboratory results, both the generated wind flow and the building pressure, were found to be in a very good agreement with the field data. However, for cornering winds, the laboratory point peak suctions at some locations on external surface near the roof edges, in the roof corner region, were markedly lower than their field counterparts. Various hypotheses were proposed and tested to clarify this problem. However, so far no definite resolution of this discrepancy was found. One of the postulated hypotheses, not tested yet, have been the Reynolds number effects and inability to correctly model a fine structure of the "delta-type" vortices in the roof corner region, during testing in a boundary-layer wind tunnel of a conventional size.

The above studies were performed in long-test section boundary-layer wind tunnels of a moderate (1.8 m by 1.8 m) cross section. Flow blockage effects limited the geometrical scales and the size of models employed in testing. A natural extension of the above study would be a companion laboratory investigation performed in a wind tunnel of a markedly larger size. This would allow for the use of significantly larger models needed to achieve Reynolds numbers sufficiently higher than those tested in previous experiments.

A collaborative US-Japan research addressing the issue of the Reynolds number effects on wind loading on low rise buildings is proposed. A number of large wind tunnel facilities (for wind engineering applications) have been constructed over the years in Japan. Perhaps arrangements could be made to make possible an experimental investigation (employing large models) in one of these facilities. Such a collaborative study would be very unique world-wide. The results of this investigation would be of interest not only to researchers and practitioners involved in modeling of wind effects on buildings and structures, but also to professionals in other disciplines of applied/environmental fluid mechanics.

The importance of high-quality laboratory data, resulting from refined wind-tunnel modeling, has been recognized not only in context of code specifications of wind loading. The quality-control issues of the wind-tunnel results have received heightened attention also due to the fact that such data have been used in comparative investigations involving field results and in benchmark validation studies of CFD codes developed for wind engineering applications.

Other aspects related to improved modeling of wind loading on low-rise buildings could be also addressed. Possible topics might be selected from a list of those identified/investigated through

CPWE or suggested by the Japanese side.

CASE STUDY OF LONG-SPAN BRIDGE AERODYNAMICS

The ultimate validation of predictions of wind-induced effects on buildings, bridges and other structures are field studies, where the response of a constructed object is monitored over a sufficiently long period of time. Due to a number of factors, such studies are not undertaken frequently and as a result the field data base is limited. One of structures in the U.S., whose performance has been monitored for a relatively long time, is the Deer-Isle Sedgwick Suspension Bridge (DISB).

The original design of the DISB was not aerodynamically efficient and it led to excessive vibration of the deck, observed in moderate winds. The deck has been stiffened by adding stay cables, but this modification did not improve the bridge aerodynamic performance. This prompted establishment of a research program supported by US Federal Highway Administration (FHWA) and geared towards development of a retrofit scheme to mitigate wind effects on the DISB. As a result of laboratory testing and analytical studies, a retrofit scheme consisting of bridge deck fairings has been developed and installed on the DISB. In parallel, the bridge has been instrumented to monitor the bridge aerodynamic response and approach wind.

The field measurements of the deck response confirmed overall effectiveness of the retrofit. Measurable excursions of the bridge deck have been observed since then. However, they occurred very infrequently. In order to further map the aerodynamic performance of the DISB, the field monitoring program has been continued and it is presently operational. Instrumentation employed in this study has been upgraded over the years. Recently, a video camera system has been installed to allow for remote visual inspection of the bridge site and instrumentation. The cataloged data spans over twenty years and it consists of records of approach wind velocity and the response of the bridge deck and one of the cable-supporting towers.

In view of the available field data and considerable previous laboratory and analytical effort, the DISB appears to be an ideal candidate for a continued case study, addressing issues of aerodynamics of moderate and long-span suspension bridges. And it is proposed as a possible subject of collaborative US-Japan investigation to be undertaken under auspices of UJNR. All the laboratory investigations of the DISB (undertaken so far) have employed section models. A natural extension of this work would be a wind-tunnel study of the DISB using a full aeroelastic model. A moderate size aeroelastic model, could be jointly designed and tested both in the US and in Japan, in standard boundary-layer wind tunnel(s). The experimental results could be then compared with field measurements and analytical results. As a possible next phase, a larger in size aeroelastic model of the DISB could be designed, manufactured (e.g. in the US), and then tested in one of large wind-tunnel facilities existing in Japan. FHWA appears to be interested in participation in work on this topic (Bosch, 1999).

CONCLUDING REMARK

It is suggested that a coordinated collaborative US-Japan activities in wind engineering have a high potential to create "added value" in outcome of the carried out effort and to increase societal recognition in both the countries of benefits of fundamental and applied wind engineering research.

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Session 3 Wind-induced Response

(Chaired by Y. Tamura)

- (1) Long-span bridge response to wind: Recent developments.

(N. Jones, R. Scanlan and H. Katsuchi)

- (2) Wind-induced response characteristics of bridges in the wake of a hill.

(K. Kimura, U. Roy and Y. Fujino)

- (3) Effect of turbulence on the space-time characteristics of the self-excited forces on a rectangular cross-section.

(F. Haan, A. Kareem and A. Szewczyk)

- (4) Cooperative study between Public Works Research Institute and Honshu-Shikoku Bridge Authority on wind resistant design of super long-span bridges.

(H. Sato and N. Furuya)

- (5) Development of computational models for bridge deck motion.

(T. Mabuchi, H. Sato, D. Reed, G. Turkiyyah and H. Yeh)

- (6) Experimental study on reducing aerodynamic response of tall buildings with open passage design.

(H. Kikitsu)

- (7) Wind-induced dynamic behavior of a single-layer latticed dome.

(Y. Uematsu, A. Sasaki and M. Yamada)

4. The fourth part of the document discusses the implications of the findings. It suggests that the results have significant implications for the field of study and may lead to further research in this area.

5. The fifth part of the document concludes the study. It summarizes the main findings and provides a final statement on the importance of the research.

Long-Span Bridge Response to Wind: Recent Developments

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ABSTRACT

The maximum span of long-span bridges has been extended in recent decades to where today, the Akashi-Kaikyo Bridge (1991 m) has been completed, and longer bridges are planned (e.g., Messina Straits (3300 m), Gibraltar Straits (5000m)). These successes are due in particular to progress in wind-resistant design; a primary component in the design of long-span bridges. Recently, multi-mode flutter and buffeting analysis procedures have been developed. These procedures, which were based on frequency-domain methods, take into account the fully coupled aeroelastic and aerodynamic response of long-span bridges to wind excitation. The focus in this brief paper is to outline recent developments of this theory and presentation of the results of an example comparing the analytical predictions with full-bridge-model wind tunnel data for the Akashi-Kaikyo Bridge.

INTRODUCTION

The current methodology for the estimation of the response of long-span bridges to wind loads incorporates the following components:

Measurement of a comprehensive set of aerodynamic and aeroelastic parameters for a given cross section using a suitably (i.e., aerodynamically) scaled section model. These parameters include: the static coefficients (lift, moment, and drag at a number of different angles of attack) and the flutter derivatives, generally also at several positive and negative angles of attack. It is emphasized that these quantities are intended to be *sectional quantities* that will be used in the analytical model.

A detailed numerical (generally finite element) dynamic model for the bridge under consideration. This model will be expected to provide a set of eigenvalues and eigenvectors for the structure and a corresponding set of generalized inertias. Generally, this will include at least 20 modes, but in some cases more may be required.

An analytical framework and computational aids for synthesizing the above data.

In addition to the above components, knowledge or assumptions about modal damping values and the wind environment are also required.

BASIC FORMULATION

Single-mode flutter and buffeting analysis procedures along with their underlying assumptions and philosophies were outlined in detail in Scanlan and Jones (1990) and elsewhere. Only the core equations are retained herein, and the single-mode formulation is replaced by the fully coupled representation.

The lift, drag, and moment per unit span are assumed to be separable into motion-dependent (aeroelastic) and motion-independent (buffeting) components. The aeroelastic or self-excited forces may then be represented as

$$L_{ae} = \frac{1}{2}\rho U^2 B \left[K H_1^* \frac{\dot{h}}{U} + K H_2^* \frac{B \dot{\alpha}}{U} + K^2 H_3^* \alpha + K^2 H_4^* \frac{h}{B} + K H_5^* \frac{\dot{p}}{U} + K^2 H_6^* \frac{p}{B} \right] \quad (1)$$

$$D_{ae} = \frac{1}{2}\rho U^2 B [K P_1^* \frac{\dot{p}}{U} + K P_2^* \frac{B\dot{\alpha}}{U} + K^2 P_3^* \alpha + K^2 P_4^* \frac{p}{B} + K P_5^* \frac{\dot{h}}{U} + K^2 P_6^* \frac{h}{B}] \quad (2)$$

$$M_{ae} = \frac{1}{2}\rho U^2 B^2 [K A_1^* \frac{\dot{h}}{U} + K A_2^* \frac{B\dot{\alpha}}{U} + K^2 A_3^* \alpha + K^2 A_4^* \frac{h}{B} + K A_5^* \frac{\dot{p}}{U} + K^2 A_6^* \frac{p}{B}] \quad (3)$$

where ρ is the air density, U the mean velocity of the oncoming wind (which is turbulent in general), $K(= B\omega/U)$ is the reduced frequency, ω the circular frequency and $H_i^*, P_i^*, A_i^*, i = 1, \dots, 6$ are functions of K and are the experimentally determined flutter derivatives for the deck cross section under investigation.

The sectional buffeting lift, moment and drag forces are expressed in terms of steady average lift, twist and drag force coefficients C_L, C_M and C_D , respectively, (referred to deck width B) of a typical deck section as (exclusive of the steady force component)

$$L_b = \frac{1}{2}\rho U^2 B \left[C_L \left(2 \frac{u}{U} \right) + (C'_L + C_D) \frac{w}{U} \right] = \frac{1}{2}\rho U^2 B L_b(x, t) \quad (4)$$

$$D_b = \frac{1}{2}\rho U^2 B \left[C_D \left(2 \frac{u}{U} \right) + C'_D \frac{w}{U} \right] = \frac{1}{2}\rho U^2 B D_b(x, t) \quad (5)$$

$$M_b = \frac{1}{2}\rho U^2 B^2 \left[C_M \left(2 \frac{u}{U} \right) + C'_M \frac{w}{U} \right] = \frac{1}{2}\rho U^2 B^2 M_b(x, t) \quad (6)$$

The system of equations can be written (after Fourier transformation into the reduced frequency (K) domain) (Scanlan and Jones 1990; Jain et al. 1996; Jain 1996)

$$E \bar{\xi} = \bar{Q}_b \quad (7)$$

where $\bar{\xi}$ and $\bar{Q}_b$ are the Fourier-transformed ξ (generalized coordinate) and Q_b (generalized force) vectors, respectively. A general term of the impedance matrix E is

$$E_{ij} = -K^2 \delta_{ij} + iK A_{ij}(K) + B_{ij}(K); \quad i = \sqrt{-1} \quad (8)$$

with terms of matrices A and B , and vector Q_b expressed as

$$A_{ij}(K) = 2\zeta_i K_i \delta_{ij} - \frac{\rho B^4 \ell K}{2I_i} [H_1^* G_{h_i h_j} + H_2^* G_{h_i \alpha_j} + H_5^* G_{h_i p_j} + P_1^* G_{p_i p_j} + P_2^* G_{p_i \alpha_j} + P_5^* G_{p_i h_j} + A_1^* G_{\alpha_i h_j} + A_2^* G_{\alpha_i \alpha_j} + A_5^* G_{\alpha_i p_j}] \quad (9)$$

$$B_{ij}(K) = K_i^2 \delta_{ij} - \frac{\rho B^4 \ell K^2}{2I_i} [H_3^* G_{h_i \alpha_j} + H_4^* G_{h_i h_j} + H_6^* G_{h_i p_j} + P_3^* G_{p_i \alpha_j} + P_4^* G_{p_i p_j} + P_6^* G_{p_i h_j} + A_3^* G_{\alpha_i \alpha_j} + A_4^* G_{\alpha_i h_j} + A_6^* G_{\alpha_i p_j}] \quad (10)$$

$$Q_{bi}(x, s) = \frac{\rho B^4 \ell}{2I_i} \int_0^\ell \{L_b(x, s) h_i + D_b(x, s) p_i + M_b(x, s) \alpha_i\} \frac{dx}{\ell} \quad (11)$$

where δ_{ij} is the Kronecker delta function. The modal integrals ($G_{r_i s_j}$) are computed as

$$G_{r_i s_j} = \int_0^\ell r_i(x) s_j(x) \frac{dx}{\ell}; \quad r_m, s_m = h_m, p_m, \alpha_m \quad (12)$$

As evident from Eqs. 9 and 10, the degree of coupling depends strongly on the structural mode shapes – through the $G_{r_i s_j}$ integrals – and the numerical value of the flutter derivatives, both of which directly influence the magnitude of the off-diagonal terms.

Flutter is identified by solving the aeroelastically influenced eigenvalue problem with $\bar{Q}_b = 0$ in Eqn. 7. To obtain a non-trivial solution for this equation, the determinant of matrix E must vanish. Determining the values of K and ω for which both the real and imaginary parts of the determinant are zero identifies the flutter condition (Jain et al. 1996). The non-trivial solution of the eigenvalue problem also yields a non-zero $\bar{\xi}$ vector that indicates the relative participation of each structural mode in the flutter condition.

To estimate the buffeting response, the power spectral density matrix for the generalized displacements ξ is developed using Eqn. 7 as

$$S_{\xi\xi}(K) = E^{-1}S_{Q_bQ_b}[E^*]^{-1} \quad (13)$$

where E^* is the complex conjugate transpose of matrix E .

The PSD and corresponding estimates of the mean-square values of the physical displacements can be obtained from the PSD of the respective generalized displacement components through, for example,

$$S_{hh}(x_A, x_B, K) = \sum_{i,j} B^2 h_i(x_A) h_j(x_B) S_{\xi_i \xi_j}(K) \quad (14)$$

$$\sigma_{hh}^2(x_A, x_B) = \int_0^\infty S_{hh}(x_A, x_B, n) dn \quad (15)$$

THE AKASHI-KAIKYO BRIDGE MODEL

The first 25 vibration modes of the Akashi-Kaikyo Bridge were used for the flutter analysis, and 17 modes for the buffeting analysis. The predicted and measured flutter speeds for the original cross section at zero-degree angle of attack were 79.1 m/s and 84.0 m/s, respectively, while for the modified, aerodynamically superior, cross section were 81.3 m/s and 90.0 m/s, respectively. In all cases considered, mode 12 (1st symmetric torsion) is the leading flutter mode. All of the flutter speeds computed were in the range 4% - 11% of those measured in the wind tunnel full-bridge model, and are in good agreement with those predicted independently in Japan.

Fig. 1 shows the comparisons of the buffeting responses between the multi-mode, SRSS responses, and measurements (HSBA 1992) for the original cross section at midspan (figures for other locations and cross sections are available in Katsuchi (1997)). Good agreement exists between the multi-mode responses and measurements in the vertical and torsional directions. In particular, the multi-mode calculation could well capture the feature of the measurements that the torsional response rapidly increased after 70 m/s.

Upon examination, it is evident that the difference between the torsional response of the multi-mode and SRSS responses only becomes large as the wind speed increases, indicating that coupling effects in the multi-mode calculation become important. Overall, however, the SRSS approach gives good agreement. A difference of a factor of about two persisted in the lateral response calculation, even though the analysis took into account every parameter which was measured from the wind tunnel and the model; this issue remains a focus of ongoing investigation. Its resolution may ultimately require measurement data from the full-scale structure. Overall, the agreement between the analytical estimates and the wind tunnel data are considered excellent in both flutter and buffeting.

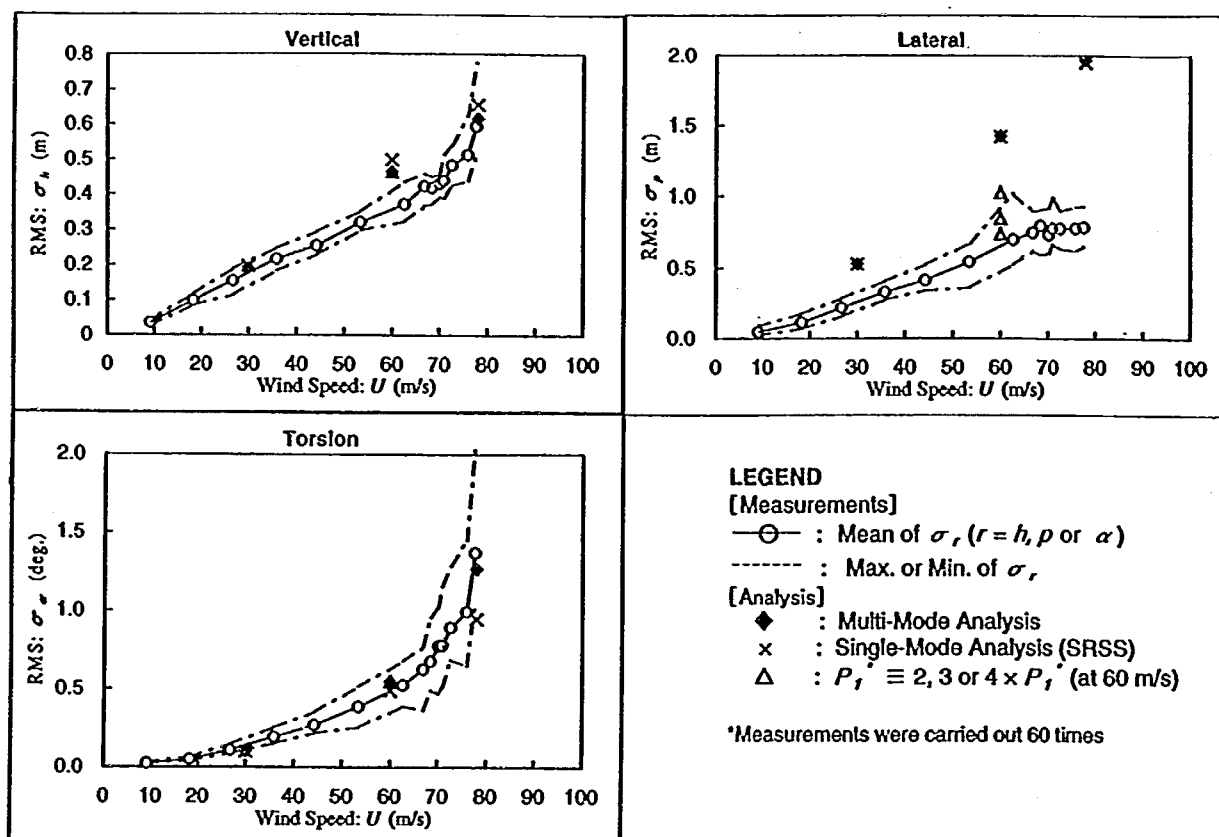


Figure 1: Buffeting response estimates - original cross section

CONCLUDING COMMENTS

An overview of a state-of-the-art approach to the prediction of the response of long-span bridges to wind has been presented. While there are variations on this basic approach proposed by a number of other researchers, the method described herein represents this inherently complex problem in a manner which brings out the important mechanics in a reasonably straightforward manner. The example presented in brief attests to the ability of the methodology to capture the important features of the response.

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Wind-induced response characteristics of bridges in the wake of a hill

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ABSTRACT: The vertical response characteristics of a flat plate cantilever model, which can be thought as a simplified model of a cable stayed bridge at its erection stage, in the wake of a conical hill were experimentally studied. The response was found to be much larger and spikier than in the approaching boundary layer. The flow characteristics that caused the spiky responses were studied by conditionally sampling the velocity fluctuations based on the response spikes. It was clarified that the velocity fluctuations were highly correlated along the line skewed approximately 40° from normal to the approaching wind direction. Also the velocity fluctuation seemed to have longer period than the spiky velocity fluctuations and it seemed to be caused by the large scale flow structure in the wake of the hill.

INTRODUCTION

For long span bridges, wind-induced responses have to be carefully considered for their design. When the bridge is located in a complex terrain, its effects on the approaching flow and the response must be considered. Davenport et al. (1990) conducted a series of wind tunnel experiments using full bridge models with their complex surroundings such as steep banks of river, a dam, mountains and so on, and pointed out the importance of the effects of surrounding topography on the wind-induced responses of the bridges.

During the full bridge wind tunnel test of the Tataara Bridge, large horizontal, and large and spiky vertical responses, which were not recognized previously, were observed (Miyata et al. 1996). These unusual response characteristics were attributed to the oncoming flow that is largely affected by a hill of 472 m high located 1 km away from the bridge. However the causes of this kind of unusual response characteristics of the bridge in the wake of a hill are not clear.

The objective of this study is to clarify the unusual response characteristics in the wake of a hill using a flat plate cantilever bridge model that can be thought as a very much simplified version of a cable stayed bridge at its erection stage. Also the cause of the spiky vertical response was attempted to be correlated with the velocity fluctuation characteristics.

EXPERIMENTAL SETUP

The size of the bridge and hill models was designed based on a scale of 1:2000 of the Tataara Bridge case. A conical hill with height of 20 cm and diameter of 90 cm was used. The model hill was covered with woolen cloth to increase the surface roughness. A flat plate cantilever made of white cedar with length, width and thickness of 223 mm, 16 mm and 0.5 mm, respectively, was used as the bridge model. The flat plate model was supported at the height of 23 mm and was allowed the vertical vibration. The first natural frequency and critical damping ratio of the bridge model were 7.2 Hz and 2.4%, respectively.

The experiments were conducted using a boundary layer wind tunnel of the Department of Civil Engineering, University of Tokyo. The wind tunnel is a vertically located closed circuit type and has a 1.5 m wide, 1.8 m high and 11 m long closed test section. A boundary layer was generated by a small fence located near the entrance of the test section and steel chains set on the floor of the test section. The mean velocity profile seems to fit the power law with an exponent $\alpha = 0.13$ and the boundary layer thickness $\delta = 55$ cm.

The model hill was placed in the generated boundary layer with the hill center at the center of the tunnel width. Figure 1 shows the definition of the co-ordinates of the model locations and the wind yaw angle, β , of the model. Also the model tip locations where the measurements were taken are shown with black dots.

The tip of the bridge model was set at each measurement location with different horizontal wind yaw angles, β . For convenience, the response measured with wind yaw angles of -160° , 180° , 160° , 140° , and 120° at the positive y -coordinate locations will be presented as the response measured at the symmetric locations with negative y -coordinate at angles of -20° , 0° , 20° , 40° , 60° , respectively, based on the symmetric flow characteristics.

The vertical response of the bridge model tip at $U_g = 1.6$ m/s was measured by a laser displacement sensor fixed at the position 35 cm above the bridge model. The response was taken after low-pass filtered at 20 Hz cut-off frequency with sampling frequency of 50 Hz and the sampling time of 5 min.

EXPERIMENTAL RESULTS

Figure 2 shows the r.m.s. of the bridge model tip deflection at different locations. The model response measured in the boundary layer without the hill at the same gradient mean wind speed $U_g = 1.6$ m/s is also shown in the figure as BL. With respect to the different x -coordinates, the response becomes largest at $x = 80$ cm.

The effects of horizontally skewed wind on the r.m.s. response are presented in Figure 3 (model located at $x = 80$ cm): There was a clear tendency in the response magnitudes that the response becomes larger with $\beta = 40^\circ$ than with the other wind yaw angles at the locations with positive y -coordinates. On the other hand, at the locations with negative y -coordinates, the response became larger with $\beta = -20^\circ$. The same tendency was also observed at other x -coordinate locations.

The spikiness can be expressed in terms of coefficient of kurtosis (C_4) which is the fourth central moment normalized by the standard deviation and it takes the value of 3 for Gaussian process. In the boundary layer, the coefficient of kurtosis was 3.4 whereas in the wake of the hill was much larger. Figure 4 shows the coefficient of kurtosis for different x - and y -coordinate locations with, $\beta = 0^\circ$. The coefficients of kurtosis are generally larger around $y = 25$ and -15 cm locations, but the tendency is not very regular. Similar tendency was observed with most of other yaw angles but the effect of wind yaw angles on the spikiness of the response was not clear as on the response magnitude.

CONDITIONAL SAMPLING OF VELOCITY FLUCTUATIONS BASED ON THE RESPONSE SPIKES

The response spikes were identified by the amplitudes that exceed $k\sigma$, where σ represents the standard deviations of the whole time series and k is a threshold level that was taken as 3 in this study. It was observed that in the wake of the hill, downward spikes were more dominant than the upward spikes and, therefore, the characteristics of velocity fluctuations corresponding to the downward spikes will be discussed here.

Simultaneous measurement of the bridge model response and the vertical component of the wind velocity fluctuations at two locations near the bridge model was conducted at different yaw angles of $\beta = 0^\circ$ and $\beta = 40^\circ$ with the model tip located at $x = 80$ cm and $y = 15$ cm. The measurements were repeated with different distances between the velocity measurement probes, keeping one probe M2 at 1 cm windward of the model tip, and changing the location of the other probe M1 parallel to the model axis. The sampling frequency and time were 1 kHz and 5 min, respectively. For each measurement, approximately 60 to 70 downward spikes were identified and the velocity fluctuations were conditionally sampled based on the response spikes. The identified response spikes and the retrieved velocity fluctuations were averaged and their peak value P_R , P_{M1} and P_{M2} were obtained. The normalized value of P_{M1}/P_R by P_{M2}/P_R is plotted against the distance between two probes in Figure 5. In the wake of the hill, the velocity fluctuation that corresponds to the response spikes were more correlated along $\beta = 40^\circ$, whereas in the boundary layer the correlation was more or less the same for both yaw angle cases. This seems to explain the cause of the effects of the wind yaw angle on the response magnitude shown in Figure 3.

The time history of the velocity fluctuations corresponding to the response spikes was studied to clarify whether the spiky velocity fluctuation causes the response spike or not. It was found that about 80% of the identified response spikes did not correspond to the velocity fluctuation spikes. This characteristics can more clearly be observed by studying the average velocity fluctuation time history that was conditionally sampled with the response spikes. Figure 6 shows the averaged response spikes and corresponding velocity fluctuations in the wake of the hill and in the approaching boundary layer. The velocity fluctuation that caused the response spikes seemed to fluctuate with the natural period of the bridge model that was much larger than the period of spiky velocity fluctuations. A large scale flow structure behind the hill seems to be the cause of this kind of velocity fluctuations.

CONCLUSIONS

1. Larger and spikier responses behind the hill than in boundary layer at the same gradient wind speed were observed.
2. The response magnitude strongly depended on the wind yaw angle. The response due to the wind with yaw angle around 40° produced the largest response for positive y-coordinate locations and that with approximately -20° for negative y-coordinate locations. This may be explained by the larger correlation of the velocity fluctuations along the line with the same wind yaw angle that caused the spiky responses.
3. The velocity fluctuation that caused spiky responses was not necessarily spiky, but it fluctuated with the natural period of the bridge model. Such velocity fluctuation seemed to be caused by a large scale flow structure in the wake of the hill.

This study was partly supported by the Honshu-Shikoku Bridge Authority and it is gratefully acknowledged.

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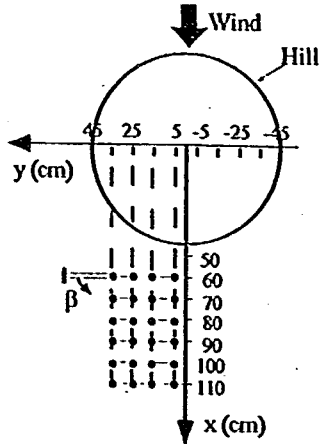


Figure 1. Coordinate definition.

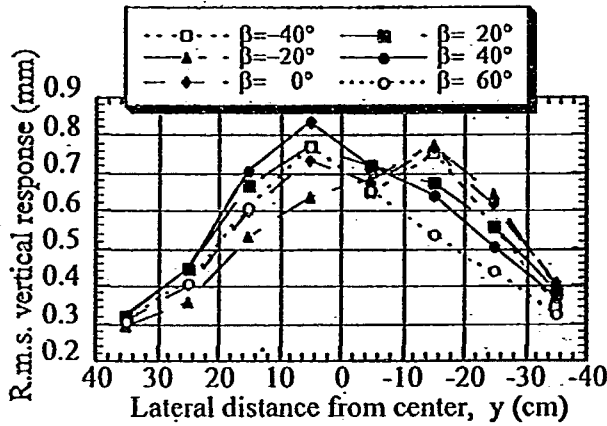
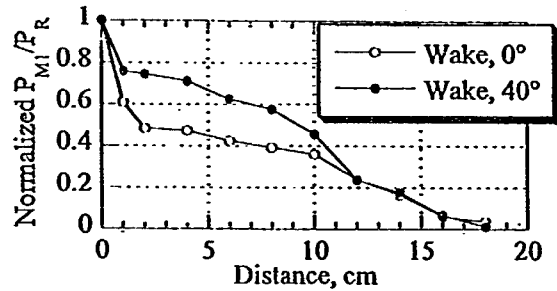
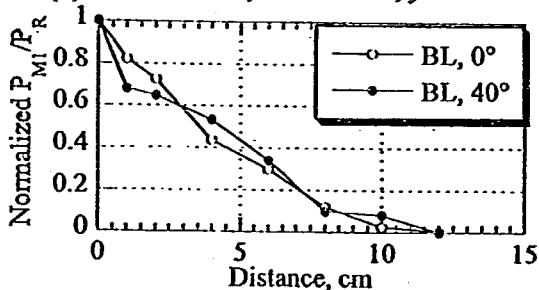


Figure 3. R.m.s. vertical response at different yaw angles ($x = 80$ cm).



(a) in the wake, $x = 80$ cm, $y = 15$ cm



(b) in the boundary layer

Figure 5. Correlation of velocity fluctuations corresponding to response spikes.

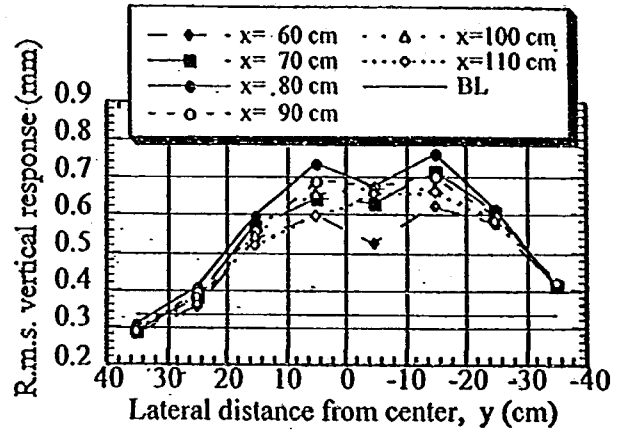


Figure 2. R.m.s. vertical response at various locations ($\beta = 0^\circ$).

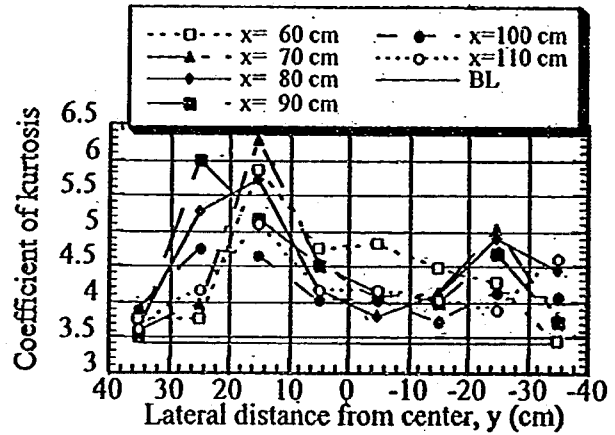
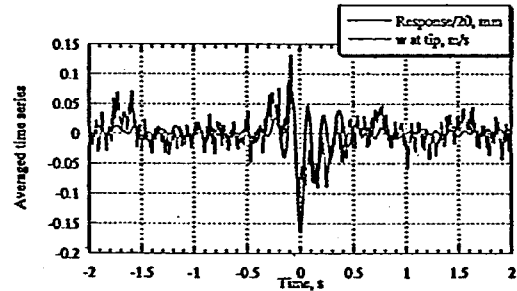
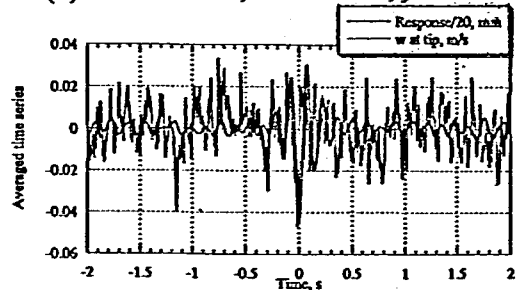


Figure 4. Coefficient of kurtosis at various locations ($\beta = 0^\circ$).



(a) in the wake, $x = 80$ cm, $y = 15$ cm



(b) in the boundary layer

Figure 6. Averaged response spike and corresponding velocity fluctuation

Effect of Turbulence on the Space-Time Characteristics of the Self-Excited Forces on a Rectangular Cross Section

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ABSTRACT

The complexity of flow-structure interactions requires that wind-tunnel testing be an integral element of the design process for long, suspended-span bridges. This complexity coupled with the fact that only a fraction of the turbulent characteristics of the atmosphere can be matched in wind tunnels requires designers to understand the effects of imperfect turbulence simulation as fully as possible. This study addresses the effects that turbulence has on the "self-excited" or "motion-induced" forces responsible for bridge flutter—more specifically, on the flow mechanisms responsible for these effects.

INTRODUCTION

Complexity in the interaction between long span bridges and atmospheric flows requires bridge designers to test models in wind tunnels. Given the fact that atmospheric flows cannot be modeled perfectly in experiments, an understanding of turbulence effects is crucial. To study turbulence effects on the self-excited forces on bridge decks, a wind-tunnel model of rectangular cross section was considered as a starting point. To allow a study of the mechanisms of these effects, surface pressure measurements were made employing the approach used by Matsumoto et al. (1995). While these distributions can be integrated to provide conventional flutter derivatives, they also provide insight into the mechanisms involved in the turbulence effects on flutter.

Amplitude and phase distributions for the surface pressures are defined according to the following equations. For torsional motion described in terms of the angular position, α , in equation (1), each streamwise position along the model had a pressure fluctuation at the oscillation frequency described by equation (2).

$$\alpha(t) = \alpha_0 \cos(2\pi f_m t) \quad (1)$$

$$C_p(x, t) = -C_p^*(x) \cos(2\pi f_m t - \psi(x)) \quad (2)$$

where α_0 is the oscillation amplitude, f_m is the model oscillation frequency, C_p is the pressure fluctuation (normalized by the dynamic pressure) at frequency f_m , C_p^* is the amplitude and ψ is the phase of this fluctuation as a function of streamwise position, x . Self-excited lift and moment were expressed according to equations (3).

$$L = q2b \left[kH_2^* b \frac{\dot{\alpha}}{U} + k^2 H_3^* \alpha \right] \quad M = q(2b^2) \left[kA_2^* b \frac{\dot{\alpha}}{U} + k^2 A_3^* \alpha \right] \quad (3)$$

where q is the dynamic pressure, b is half the deck width, $k = 2\pi f_m b / U$, U is the mean longitudinal velocity, and α is the angular position of the model (and the dot denotes differentiation with respect to time). Flutter derivatives, H_i^* and A_i^* , were derived from relations similar to those used in Matsumoto et al. (1995).

EXPERIMENTAL SETUP

All work reported here was conducted in the 1.52 m by 1.52 m test section of the University of Notre Dame's atmospheric wind tunnel. Turbulence was produced with grids of different sizes at various upstream distances from the model. This study employed a model of rectangular cross section with frontal height, D , of 38.1 mm, a streamwise width, B , of 254 mm, and a cross-stream span, L , of 1067 mm. A servomotor mechanism was used to force the model in a pitching motion with a torsional amplitude of $\pm 2^\circ$. All tests were run at a constant Reynolds number ($Re = 1.2 \times 10^4$), while reduced velocities were varied by changing the model oscillation frequency. Sixty-four SenSym pressure transducers were mounted within the model and could be connected to any two chordwise rows of taps at a given time.

RESULTS

Turbulence parameters for each of the flows used in this project are listed in Table 1. These include the longitudinal and vertical turbulence intensities (I_u and I_w , respectively), the longitudinal integral length scale of the longitudinal velocity fluctuations (L_{ux}), and the small-scale spectral density parameter (S). First suggested by Melbourne (1979), this parameter is essentially a *scale-specific* turbulence intensity used here to monitor the energy of scales of $D/10$.

The pressure amplitude distributions for each flow case of Table 1 are shown in Figures 1-3. A basic hump shape that shifts downstream with increasing reduced velocity, U_r , characterizes the smooth flow distribution of Figure 1. This same general shape is present in each of the turbulent cases of Figures 2 and 3 except that it shifts upstream with increasing turbulence intensity. The effect of smaller integral scales (the "a" cases) appears to be to lower amplitudes and to reduce the downstream extent of the hump shape. The pressure phase distributions for each flow case are shown in Figures 4-6. The smooth flow phase distribution of Figure 4 suggests the presence of a flow structure that dilates as the reduced velocity increases and contracts as turbulence intensity increases. Turbulence scale did not change the phase distributions significantly.

Sample flutter derivatives calculated from these amplitude and phase distributions are shown in Figures 7 and 8. Each turbulence case had a strong stabilizing effect on A_2^* . The flows with I_u of 12% caused a slightly greater shift from the smooth-flow values. As demonstrated in Chen et al. (1999), the shift in H_3^* has a stabilizing effect in coupled, multi-mode flutter problems. Scale effects were most evident for the H_3^* flutter derivative.

From lift and moment time series integrated from pressures measured at two spanwise positions, correlation coefficient functions were calculated. Correlation coefficients for moment are shown in Figure 9 for four different spanwise separations and each flow case. In each flow studied, the lift forces maintained a greater spanwise correlation than the moments. Case 3b—the flow having the largest turbulence intensity and the largest length scale—reduced the spanwise correlation the most for both lift and moment. In every case, the correlations were strong for spanwise separations much larger than the integral scales of the flows. Further work will involve gaining understanding of the causes of the observed spanwise correlation trends and developing methods to apply the data to full-bridge flutter predictions.

CONCLUSIONS

Pressure measurements revealed streamwise phase structures that contracted toward the leading edge with increasing turbulence intensity. The scales of the turbulent flow also altered the shape of these structures. Lift and moment were highly correlated over spanwise distances significantly larger than any scales present in the flows. Future work will focus on the physical interpretations of the effects of turbulence on the pressure amplitude and phase distributions and how they relate to the observed changes in the flutter derivatives. These flutter derivative changes in turbulence will be combined with the spanwise correlation behavior into full-bridge flutter predictions.

ACKNOWLEDGEMENTS

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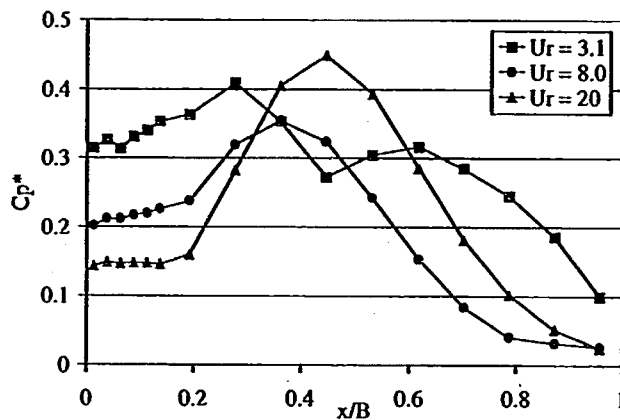


Figure 1. Pressure amplitude distributions for smooth flow.

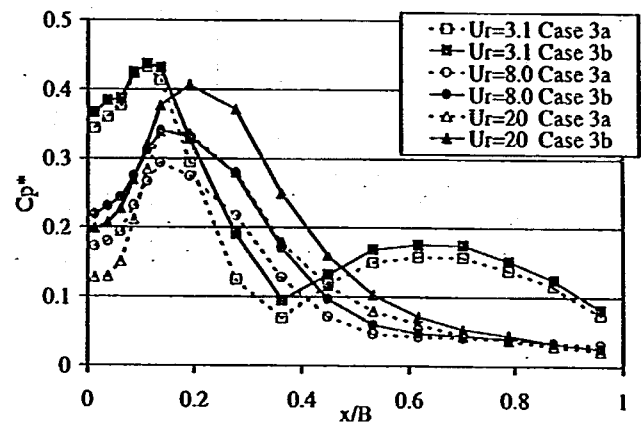


Figure 3. Pressure amplitude distributions for turbulent flows with $I_u=12\%$. Open symbols correspond to the cases with smaller L_{ux} .

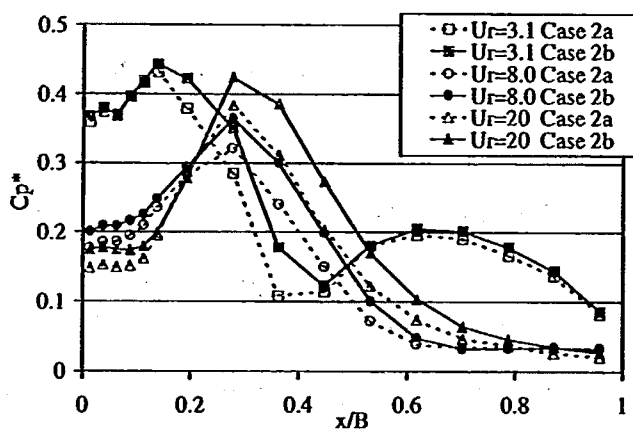


Figure 2. Pressure amplitude distributions for turbulent flows with $I_u=6\%$. Open symbols correspond to the cases with smaller L_{ux} .

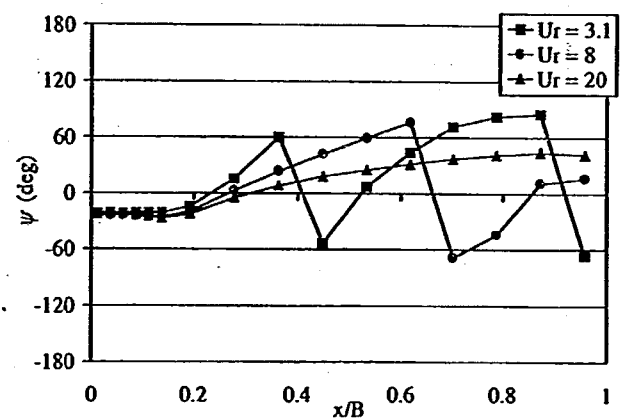


Figure 4. Pressure phase distributions for smooth flow.

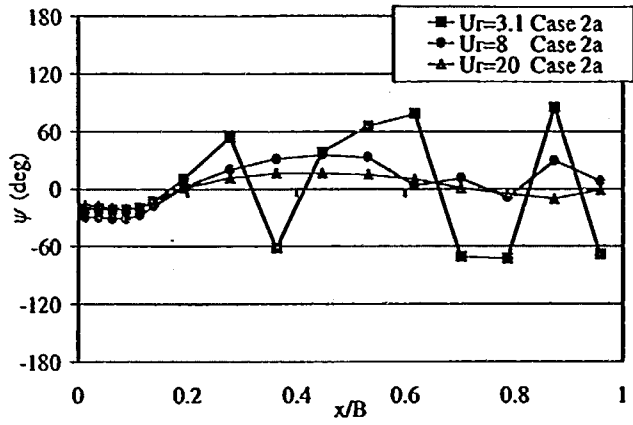


Figure 5. Pressure phase distributions for turbulent flows with $I_u=6\%$.

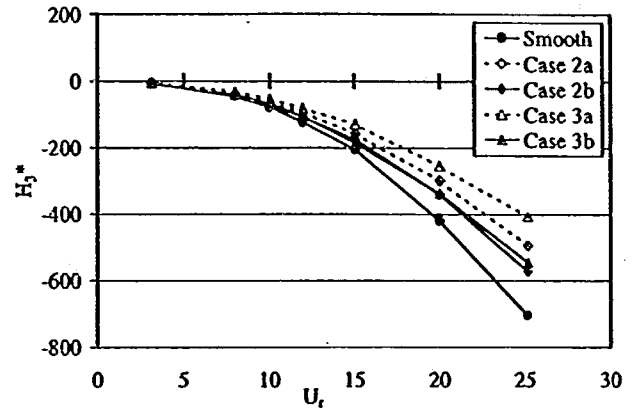


Figure 8. H_3^* flutter derivative for smooth and turbulent flow.

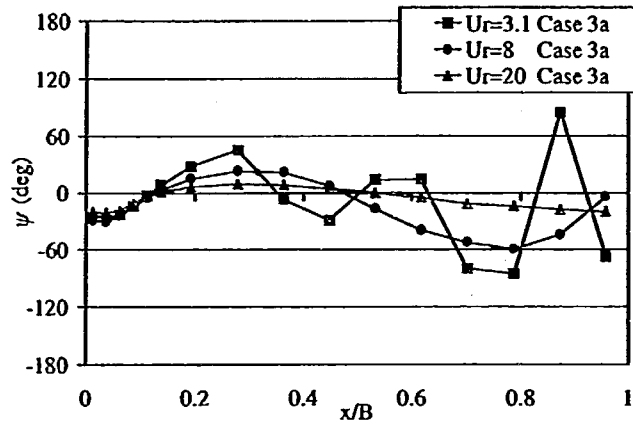


Figure 6. Pressure phase distributions for turbulent flows with $I_u=12\%$.

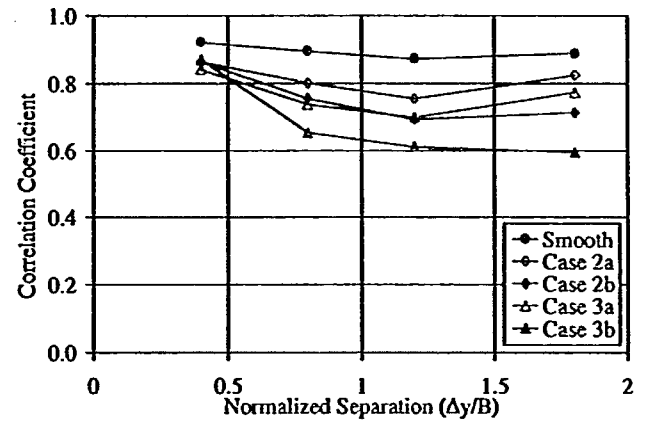


Figure 9. Correlation coefficient for lift force as a function of spanwise separation for $U_t=10$.

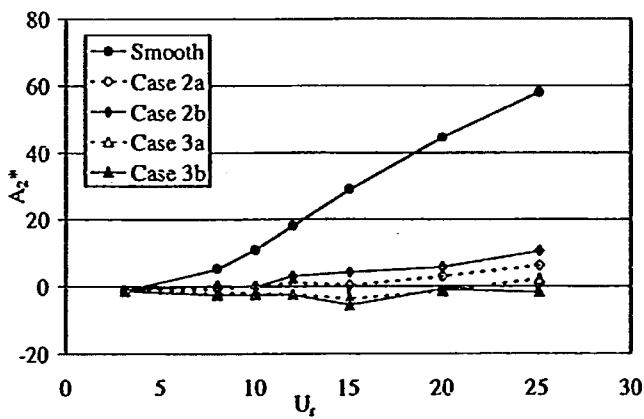


Figure 7. A_2^* flutter derivative in smooth and turbulent flow.

Table 1. Turbulent Flow Parameters

	I_u	I_w	L_{ux}	S
Smooth	0.4%	0.3%	N/A	N/A
Case 2a	6.2%	5.9%	2.0D	2.3
Case 2b	5.8%	5.1%	4.8D	0.3
Case 3a	12.0%	11.4%	1.3D	53
Case 3b	12.2%	10.2%	5.3D	6.0

Cooperative Study between Public Works Research Institute and Honshu-Shikoku Bridge Authority on Wind Resistant Design of Super Long-Span Bridges

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ABSTRACT

In this paper outlined is the cooperative study between Public Works Research Institute and Honshu-Shikoku Bridge Authority on wind resistant design of super long-span bridges. The cooperative effort started in 1989, and the Large Boundary Layer Wind Tunnel was constructed at PWRI in 1991. Using this facility, aerodynamic characteristics of the Akashi Kaikyo Bridge, the Tatara Bridge and the Kurushima Bridge were studied. Wind tunnel studies on a super long-span bridge, whose span length is longer than the Akashi Kaikyo Bridge, are now in progress.

INTRODUCTION

The Akashi Kaikyo Bridge was open to public in April, 1998. The span length of the Akashi Kaikyo Bridge is much longer than the Humber Bridge, which had been the longest bridge in the world. At the design stage of the Akashi Kaikyo bridge, it was thought that the aerodynamic stabilities of such a long-span bridge should be confirmed carefully by very accurate wind tunnel study, namely full aeroelastic model study. From this reason, cooperative study on wind resistant design of super long-span bridges was started in 1989 between Public Works Research Institute, Ministry of Construction and Honshu Shikoku Bridge Authority. The Large Boundary Layer Wind Tunnel, whose test section is 41m wide, 4m high and 30m long, was constructed at PWRI in 1991, and flutter characteristics were investigated in smooth and turbulent flow using the full aeroelastic model, whose scale ratio was 1/100.

FULL MODEL WIND TUNNEL STUDY FOR THE AKASHI KAIKYO BRIDGE[1]

Experiments in Smooth Flow

In the smooth flow test, remarkable static torsional displacement was observed. It was caused mainly by aerodynamic drag force acting to the stiffening truss. In case of ordinary suspension bridges, which have center span of about 1,000 m, static torsional displacement is negligibly small. It was found in the full aeroelastic model test of the Akashi Kaikyo Bridge, however, that relative angle of attack caused by the static displacement was not negligibly small, and that it varied along the bridge axis. Since angle of attack affects flutter characteristics, the effects of torsional displacement on the aerodynamic stability must be

considered in case of super long-span bridges, which have center span of 2,000 m or over.

Coupled flutter was observed at the wind speed of 8.5m/s (85m/s for real bridge) . The rotation center lay on the windward side at the midspan, on the leeward side at quarter point of center span, and on the windward side again at the middle of side spans. During the flutter, its vertical bending vibrational mode was not similar to any of natural modes, while its torsional vibrational mode was similar to the first symmetric natural mode. Therefore it does not seem that aerodynamic stability of super long-span suspension bridges can be predicted directly from spring-mounted rigid model test.

In order to predict flutter of super long-span bridges analytically, the effect of the static torsional displacement should be considered, and higher natural modes as well as the first natural mode should be included. Besides, aerodynamic derivatives such as Drag due to Heaving motion, Drag due to Torsional motion, Lift due to Along-wind motion, and Pitching moment due to Along-wind motion should be included in addition to the conventional aerodynamic derivatives.

Experiments in Turbulent Flow

In the turbulent flow test, gust responses were observed in horizontal bending, vertical bending and torsional mode. As wind speed increased, the vibration of torsional 1st mode became dominant, and apparent damping became smaller. This seems to be the effect of flutter. Apparent damping in the turbulent flow was compared with that in the smooth flow. It was found that the turbulence effect on flutter was small for this bridge model.

The observed gust responses were compared with the calculated ones. As for vertical bending and torsional responses, the agreement was fairly good, however, the observed horizontal bending responses were much smaller than calculated ones. One of the causes of this discrepancy was thought to be the spatial correlation which was assumed as the exponential function of $fb |x_1 - x_2| / B$. Since the measured spatial correlation of wind speed (root co-coherence function) did not tend to unity as frequency became 0 when separation of measurement points were large, the calculation might lead to an overestimation.

Using the measured aerodynamic admittance and the spatial correlation based on the turbulent flow of the wind tunnel, gust responses were calculated again. Although there still remained some discrepancy, accuracy of the calculation was improved.

FULL MODEL WIND TUNNEL STUDY FOR THE TATARA BRIDGE

Following the Akashi Kaikyo Bridge, wind tunnel studies for the Tatara Bridge and the Kurushima Bridges were also conducted at the Large Boundary Layer Wind Tunnel. The Tatara Bridge, which has the main span length of 890m, is the longest cable-stayed bridge in the world. In addition, the Tatara bridge is surrounded by islands. Some of the islands have high mountains up to 400m above the sea level. The Kurushima Bridges, which are suspension bridges with stiffening box girders, are also surrounded by islands. Therefore, the aerodynamic stabilities of these bridges were confirmed through full aeroelastic model test with topographical model.

Full Model Test for the Tatara Bridge with Island Model [2]

Wind characteristics at the site are very complicated. For example, in the case of

wind direction of NNW and SSE, a topographical effect is hardly recognizable. In other words, wind speed and turbulence intensity were scarcely changed by the topography. On the contrary, in the case of wind direction such as NE and W, wind characteristics were changed significantly by the topography. Particularly, in the case of NE direction, the turbulence intensity was considerably increased by the topography, but the wind speed was not remarkably decreased. In the design of the long-span bridge, these wind characteristics were thought to be critical.

Wind tunnel experiments were carried out for three cases of wind direction, namely 18, 36 and 180 degrees. Wind direction was defined clockwise from the north direction normal to bridge axis. When wind direction was 18 and 180 degrees, the topographical effect was hardly recognizable. On the other hand, in the case of 36 degrees, the topography increased the gust response significantly.

According to the quasi-steady theory, aerodynamic force due to approaching turbulence increases with square of mean wind speed and turbulence intensity. In the case of 36 degrees, a significant increase in turbulence intensity along the bridge axis was recognized. However, the increase in turbulence intensity was associated with decrease in mean wind speed.

Another factor that influences the gust response of long-span bridges is correlation of fluctuating wind speed at different places. The integral scale as well as intensity at the mid-point of the center span in case of 36 degree was increased very much by the topographical model. The co-coherence of fluctuating wind speed at two places on the bridge axis was much higher in case of 36 degree than in case of 180 degree.

From these measurements, it seems that the large gust response in case of wind direction 36 degree was caused by high turbulence intensity and high space-wise correlation of the turbulent flow generated by the topographical model.

STUDY ON FLUTTER CHARACTERISTICS OF SLOTTED BOX GIRDERS

Following the completion of the Akashi Kaikyo Bridge in 1998, the whole Nishi-Seto Expressway, which includes the Tatara Bridge and the Kurushima Bridges, was open to public on May 1st, 1999. In Japan and in the world, there are several plans or ideas to construct bridges longer than the Akashi Kaikyo Bridge. It was proposed by the authors that slotted box girders were effective to increase flutter speeds of such super long-span bridges.

Section Model Test[3]

The effect of location and size of slot on aerodynamic characteristics was examined through section model wind tunnel tests. Considering a super long-span bridge which has center span length of 3,000m with two side spans of 1,500m, the structural conditions were assumed. Reduced mass $\mu (=m/(\rho B^2))$, m : mass per unit length, ρ : air density, B : girder width), reduced polar moment of inertia $\nu (=I/(\rho B^4))$, I : polar moment of inertia per unit length), and natural frequency ratio $\varepsilon (=f_\theta/f_z)$, f_θ : torsional natural frequency, f_z : vertical bending natural frequency) were 16, 2.1, and 2.1, respectively. From the test results, it was found that the slot at the center increased the flutter onset wind speed. It was also found that the flutter onset wind speed was increased with the width of slot at the center of the girder.

Although wide slot at the center of the girder improves flutter characteristics,

narrower slot would be preferable from the viewpoint of construction cost of towers and foundations. To improve aerodynamic characteristics, the effect of some devices was studied by section model tests. The results showed that the center barrier and guide vanes improved flutter characteristics very well. However, the flutter speed was not so high when angle of attack was -3 deg. It was found that the guard rails at the bottom deck increased the flutter speed considerably at this angle of attack.

Flutter Analysis[3]

Using all the measured coefficients of unsteady aerodynamic forces of the slotted box girder with devices, flutter analysis of a super long-span bridge was conducted. The main span length of the super long-span bridge was 2,500 m, and the side span was 1,250 m. For comparison, flutter analysis was also conducted using unsteady aerodynamic forces of a single plate derived from Theodorsen's function. The structural conditions for both cases were same. Flutter speed for the slotted box girder was as high as about 80 m/s. On the other hand, the critical wind speed for single flat plate was as low as about 40 m/s.

Full Model Test

To verify the effectiveness of the slotted box girder and the accuracy of the flutter analysis, full model study was planned for a super long-span bridge with slotted box girder. A 1/125 model of a suspension bridge, whose span length was 1,100m +2,800m +1,100m, was constructed at the Large Boundary Layer Wind Tunnel. The wind tunnel study is now in progress.

CONCLUSION

Some of the findings from the cooperative study between PWRI and HSBA were introduced in this paper. These studies confirmed wind resistance of the Akashi Kaikyo Bridge, the Tatara Bridge and the Kurushima Bridge. In addition, some unexpected insights into flutter and gust response of super long-span bridges were obtained through these full model studies. These findings also contributed to improve analytical prediction method of wind-induced response of super long-span bridges.

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DEVELOPMENT OF COMPUTATIONAL MODELS FOR BRIDGE DECK MOTION

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The sensitivity of long-span cable-supported bridges to wind conditions is well-known [e.g., Diana, et al. (1995); Ito(1987); Iwamoto and Fujino (1995); Kubo, et al. (1992;1997); Larsen (1993); Larsen and Walther (1996); Larsen (1997); Saeki, et al. (1995); Sarkar, et al. (1994); Sato, et al. (1997); Scanlan and Tomko (1971); Scanlan (1981)]. We believe three key elements in suspended-span bridge design under wind conditions are interrelated as shown in Figure 1. These are the modeling of the wind environment with particular emphasis on the accurate representation of the longitudinal and vertical velocity fluctuations; the characterization of the bridge motion, with emphasis on identifying its sensitivity to aeroelastic phenomena such as flutter, buffeting and vortex-induced vibration of the span; and the control of adverse vibration through the modification of the deck and cables, the use of damping devices, or more directly, through appendages to modify the wind environment itself.

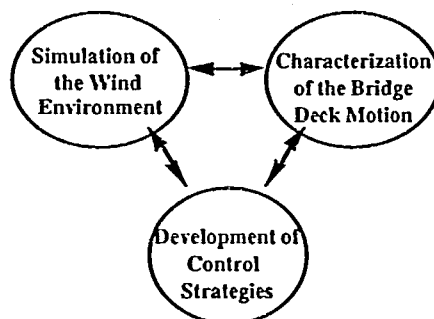


Figure 1. Key elements in the design of suspended-span bridges.

In October, 1997, the resolutions of Task Committee E stated [Golden, et al.]:

"3.6 Development of prediction methods for wind response of long-span bridges.

There are many long-span bridges in Japan and many are under construction or in the planning stages. Their sensitivity to wind effects is well-known. *Collaborative efforts to develop prediction methods for these structures based upon extensive wind tunnel and full-scale data are suggested.*" [Italics added by writers]

Along these lines, UW faculty Dorothy Reed, George Turkiyyah and Harry Yeh made a collaborative agreement with Dr. Hiroshi Sato of PWRI to investigate numerical modeling of bridge deck motion under wind conditions. Previous research carried out by Turkiyyah and Reed involving the use of the fast vortex method for two-dimensional wind simulation around a low-rise building had been successfully compared with the full-scale data for the Texas Tech Building [Turkiyyah, et al. (1995) (1997); Viozat (1995)]. Sato's research in the physical and numerical modeling of bridge deck motion is well-known [e.g., Sato(1983); Sato, et al. (1997)]. We decided to investigate the use of the

fast vortex method for bridge deck motion. An outline of our proposed collaborative process is shown in Figure 2 below:

Figure 2. Outline for the Development of Numerical Models

Initial Structural Model:
Rectangular Prism $B/D = 2$;
Extensive Wind Test Results Available from
Sato (1983): On the Aerodynamic Forces on a Rectangular Prism in Smooth and Turbulent Flow, Including Motion-Induced Effects, MS thesis, U. Western Ontario.

Drag Force:
Model at Rest, Smooth and Turbulent Flow
Aerodynamic Admittance

Lift Force:
Model at Rest, Smooth and Turbulent Flow
Vortex Excitation, Aerodynamic Admittance

Lift Force:
Model at Forced Oscillation,
Smooth and Turbulent Flow
Aerodynamic Damping

Heaving Oscillation:
Model Supported by Springs and Damper,
Smooth and Turbulent Flow:
Vortex-Induced Oscillation,
Gust Response, Galloping

Although various approaches to computational wind engineering have been investigated, we believe the discrete vortex method is well-suited to the characterization of the wind environment for the suspended-span bridge; that is, a fluid flow scenario with high Reynolds numbers and complex geometries.

Vortex methods are Lagrangian particle-based numerical simulation schemes. When coupled with fast solvers for computing vortex interactions, they become ideally suited for wind engineering simulations which tend to be characterized by high Reynolds numbers and irregular geometries. Fast vortex methods have several advantages over grid-based methods; they do not suffer from numerical diffusion; they are simpler to implement particularly as the geometry becomes more complex; and they can exploit the architecture of distributed-memory computers more effectively.

The grid-free vortex methods use the vorticity equations derived from the Navier-Stokes equations and discretize the vorticity field using a set of discrete "vortex elements". Hence, for a homogeneous fluid, the governing equations are of kinematics and the often troublesome Poisson equation for pressure is completely eliminated in the numerical scheme. The basic computational element of vortex methods is a smoothed Lagrangian element whose position and the amount of vorticity it carries are updated at each time interval. In 2-D, these elements are generally patches of

vorticity (with a circular support) called particles or blobs. In 3-D, particle representations of the vorticity field have been used [Winkelmans and Leonard (1993)], and the divergence-free vorticity condition is satisfied only approximately. The location and strengths of these computational elements determine the vorticity and therefore the velocity at any point in the domain through the Bio-Savart law [e.g. Chorin (1980)]. In numerical simulations using the vortex method, all the vortex elements are produced at the boundaries, then they are advected and diffused in 2-D flows; and in 3-D flows, the vorticity vectors are stretched by the local velocity gradients. Hence the vorticity field evolves in time based upon this computed velocity field. To account for turbulence effects, a turbulent eddy viscosity may be used. Other methods [Chorin (1990)] have also been proposed. An application of the discrete vortex method (called DVMFLOW) for bridge deck flow fields was reported by Larsen and Walther (1996).

The most time-consuming step in simulations based on vortex methods is the computation of the pairwise interaction of vortex elements needed to compute the velocity field. A direct method that evaluates all interactions is prohibitively expensive. Fast summation methods can, however, be used to significantly speed up the computations. They exhibit linear increase in time with problems size, and can be effectively parallelized on distributed-memory computers. Fast methods exploit the fact that the effect of a particle decreases with distance, and can therefore combine large numbers of elements into a small set of discrete values (on rings in 2-D and spheres in 3-D) whose effect approximates the effect of the particle cluster. Fast solvers construct and evaluate these approximations in a hierarchical fashion much like multigrid solvers. Turkiyyah and Reed (1997) examined the scalability aspects of the fast solver by developing a parallel implementation. The 3-D prototype exhibit very good speedup across a number of architectures (including both distributed memory and shared-memory machines). Speed-ups of 11 on a 16-processor Intel program were obtained. The ability to run efficiently on parallel computers is critical, because 3-D simulations may require $O(10^5)$ of vortices and are unlikely to be handled even by the fastest high-performance single-processor workstations.

We will report on our preliminary simulations involving the flow around the two-dimensional model at rest at the workshop in Tsukuba.

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Experimental Study on Reducing Aerodynamic Response of Tall Buildings with Open Passage Design

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ABSTRACT

This paper presents the results of across-wind aerodynamic responses by using several dynamic models with open passage configurations. On the whole, positive pressure going through open passages can work in effective way to reduce aerodynamic responses. As for the shallow rectangular section models, aerodynamic response is reduced in the case of along-wind open passages more than in the case of across-wind open passages. On the other hand, as for the deep rectangular section models, across-wind open passages are more effective than along-wind open passages in the lock-in phenomenon. From a point of the total damping coefficients, models with along-wind open passages tend to obtain relatively more aerodynamic damping effect and models with across-wind open passages result in having less damping coefficients.

1 INTRODUCTION

Tall buildings have a tendency to be more flexible and lighter, which means that such tall buildings are sensitive to vortex-shedding excitation and how to reduce their across-wind aerodynamics responses has been of particular interest.

This paper describes the effects of open passage configuration at appropriate positions of a tall building, which is supposed to be one of the effective approaches to reduce the effect of vortex-shedding mechanism. Open passage configuration can allow high positive pressure to flow into regions close to the side edges where vortex-shedding is generated, and also the back region of the building which has a strong relation with the across-wind dynamic force as explained by Bearman, P.W. (1967). Optimization of this design would be able to eliminate the vortex shedding system completely. Kwok, K.C.S. & Bailey, P.A. (1987) have tried various venting slots and corner treatments in an attempt to reduce wind dynamic responses. Dutton, R. & Isyumov, N. (1990) conducted a similar investigation by using various gaps, and which was discussed on the basis of the static measurement. On the other hand, Okada, H. et al (1997) introduced the effectiveness of the concept by using dynamic models. This paper discusses what effects the positive pressure going through open passages produces on the vortex-shedding mechanism and across-wind aerodynamics responses, by using several dynamic models with an aspect ratio of 8:1.

2 PROCEDURES OF WIND TUNNEL EXPERIMENTS

2.1 Wind tunnel facility

The experiments described here were performed in the Boundary Layer Wind Tunnel of the Building Research Institute, Ministry of Construction in Japan. Wind tunnel experiments were carried out in an atmospheric boundary layer flow approximately corresponding to an urban area condition which is defined as Category-3 in the A.I.J. Recommendations for Loads on Buildings (1993). The power law exponent of the mean longitudinal wind velocity profile is 0.25, and the longitudinal turbulence intensity is approximately 9.0%. The profile is generated by a set of spires and roughness blocks.

2.2 Tall building models

In this study, we prepared three types of tall building models as shown in Figure 1. All models are rigid and have an aspect ratio of 8:1. They have open passages in both along-wind and across-wind directions at the four levels, and each passage can be covered with thin acrylic resin plates. In the following discussion, the model with square cross section will be called type-1. Other two shallow rectangular section models with a side ratio of $D/B=2/3$ and $1/2$ will be called type-2 and type-3, respectively, and also two deep rectangular section models with the ratio of $D/B=3/2$ and 2 will be called type-4 and type-5, respectively. Two laser displacement sensors were mounted below the gimbal in order to record the time histories of the model deflections in both directions. The natural frequencies of the models were 14.7Hz in the across-wind direction and 24.0Hz in the along-wind direction. The structural damping coefficient of these models was 0.67% in the across-wind direction. These values were obtained from free vibration experiments.

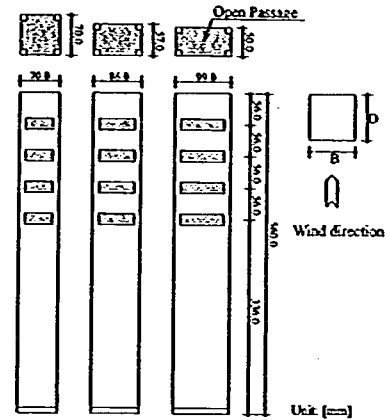


Figure 1. Configurations of Tall Building Models

3 RESULTS OF WIND TUNNEL EXPERIMENTS

3.1 Root-mean-square values of across-wind tip deflections

Figure 2 shows results of measured across-wind tip deflections under four different conditions. Vertical axis is root-mean-square values of the across-wind tip deflections normalized by the model's width D , and horizontal axis is reduced velocity $U_H/(n_0 D)$, where U_H and n_0 denote the mean wind velocity at the height of the model and the natural frequency, respectively.

As shown in Figure 2, four conditions are defined as case A, B, C and D. In the case A, all the exits of open passages are covered with acrylic resin plates. In the case B, open passages in the along-wind direction are not covered so that positive pressure can flow into the back region of the model. On the other hand, in the case C, exits of the open passages on the back are covered so that positive pressure can flow only into both of side regions. In the case D, all the exits are left to be open.

In the case of type-2 and 3 or the shallow rectangular sections, almost the same tendency can be observed in the results of both types. As seen in the results of the case B, positive pressure going through along-wind open passages works fairly effectively for reducing dynamic responses. We consider this is because positive pressure violates periodic shedding Karman vortex at the back of the model. And as for the case A and C, galloping vibration with vortex-shedding occurs in both types. But while the galloping vibration occurs at the same reduced velocity in type-2, the vibration can be controlled in type-3 by supplying positive pressure into side walls and the reduced velocity at which it starts is different each other.

Next in the case of type-4 and 5 or the deep rectangular sections, across-wind dynamic responses are generally much smaller than other cases. And positive pressure supplied into both side walls can be seen more effective than that supplied into back region. But focused on the responses around reduced velocity = 10.0, small vortex induced vibration can be seen only in the case C of both types. Such a phenomenon is generally observed in the deep rectangular section models with relatively small damping coefficient. Further discussion about this phenomenon will be in the following section.

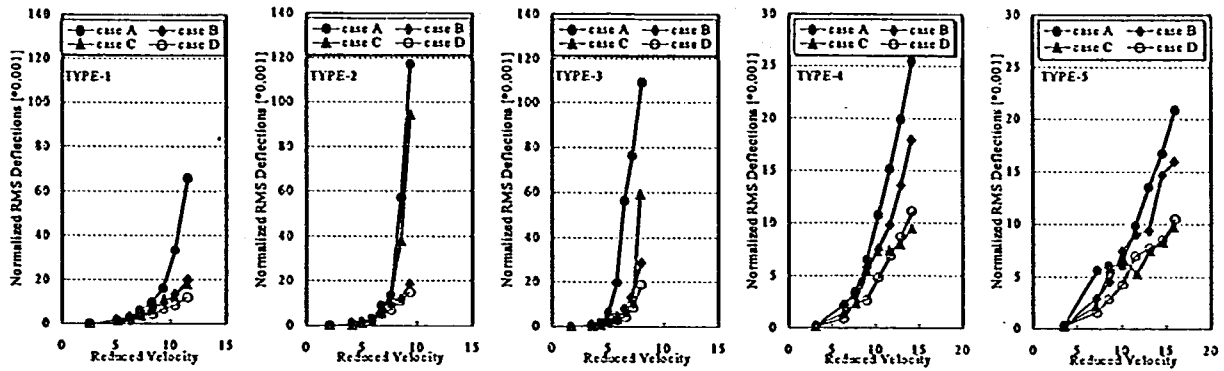


Figure 2. Normalized Root-Mean-Square Values of Across-Wind Tip Deflections

3.2 Power spectra of across-wind tip deflections

As discussed in the previous section, open passage configurations work very effectively for reducing across-wind dynamic responses. In order to evaluate relations between periodic vortex-shedding phenomena and introduced open passages, power spectra of across-wind tip deflections on type-2 are shown in Figure 3. Measured results in the figure are before the lock-in phenomenon.

As for the power spectrum of the case A or the model with no open passages, there is clear spectral peak corresponding to the vortex-shedding phenomenon. As for the case B, on the other hand, the spectral peak due to the vortex-shedding phenomenon succeeds to vanish. This result can be said to be useful effect in respect of controlling periodic vortex-shedding mechanism at the back region.

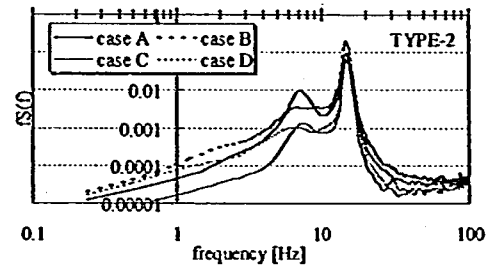


Figure 3. Measured Power Spectra of Across-Wind Tip Deflections before the Lock-In Phenomenon

3.3 Damping coefficients

In this section, from measured dynamic response data, we extracted total damping coefficients of the models that comprise of both structural and aerodynamic damping components. The coefficients were obtained by means of auto-correlation method. The total damping coefficients are plotted against simultaneously measured reduced velocity in Figure 4. Dotted line in the figure indicates the structural damping coefficient obtained from free vibration experiments.

In the case of the shallow rectangular sections, total damping coefficients tend to decrease as reduced velocity increases, while the coefficients have the tendency to increase in the case of the deep rectangular sections. These results indicate that aerodynamic component of the total damping coefficient depends on the configuration of the model's section considerably.

According to all the results, it can be generally considered that models with along-wind open passages (case B) obtain relatively more aerodynamic damping effect, and that models with open passages on side walls (case C) have less damping coefficients than others. These tendencies can be observed in every type of the side ratio D/B . As pointed out in the previous section, small vortex-shedding excitation is observed in the case C of the deep rectangular sections, which is partly because models in the case tend to have less aerodynamic damping coefficients and be relatively easy to excite. Also as for the results in the lock-in phenomenon of the shallow rectangular models, the total damping coefficients in the case C are less than structural damping coefficient itself. These results imply that

positive pressure going through both side walls doesn't always work in effective way to control the vortex shedding mechanism, although the aerodynamic response itself can be reduced.

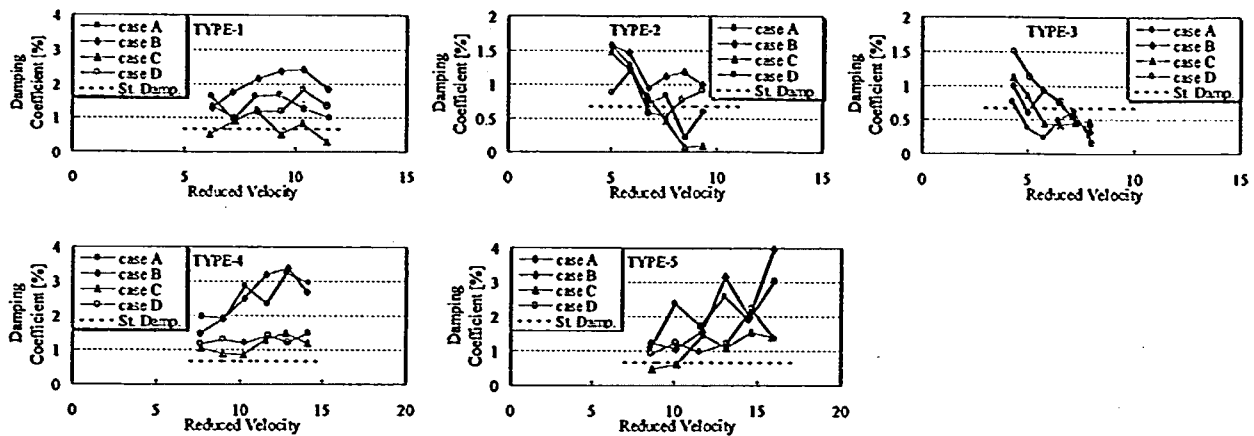


Figure 4. Total Damping Coefficients versus Reduced Velocity

4 CONCLUSIONS

By using several dynamic models with open passage configurations, across-wind aerodynamics response measurements were carried out and the following conclusions were obtained.

- (1) In the case of the shallow rectangular sections, positive pressure going through along-wind open passages works more effectively for reducing responses because of violating periodic shedding vortex in the back region.
- (2) In the case of the deep rectangular sections, positive pressure going through across-wind open passages can be seen more effective than that going through along-wind open passages in the lock-in phenomenon. But the models with across-wind open passages were observed to have small vortex induced vibration around reduced velocity = 10.0.
- (3) Positive pressure going through along-wind open passages can lead to relatively more aerodynamic damping effect. On the other hand, positive pressure going through both side walls doesn't always work in effective way to reduce aerodynamic responses. These tendencies can be observed in every type of the side ratio D/B .

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WIND-INDUCED DYNAMIC BEHAVIOR OF A SINGLE-LAYER LATTICED DOME

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ABSTRACT

This paper describes the results of some basic investigations on the wind-induced dynamic behavior and the resultant load estimation of a single-layer latticed dome with a long span. The wind pressures at many locations on dome models were measured simultaneously in a wind tunnel. The dynamic response of several domes was analyzed in the time domain, using the wind pressure data obtained from the wind tunnel experiment. The characteristics of the dynamic behavior are investigated; a special attention is paid to the vibration modes, which dominate the dynamic response. Finally, a discussion is made of the load estimation for the dome. A simple model for evaluating the design wind load is proposed.

INTRODUCTION

The subject of this study is a rigidly jointed single-layer latticed dome. Since this type of dome is light and flexible, compared with double-layer latticed domes, it tends to deflect and oscillate under turbulent wind forces. With a decrease in the natural frequencies, resulting from an increase in span, the susceptibility of the dome to resonant excitation increases. These dynamic changes should be considered in the wind resistant design.

This paper describes the results of some basic investigations aiming at achieving a better understanding of the wind-induced dynamic behavior of a rigidly-jointed single layer latticed dome with a long span. The dynamic response of several dome models is numerically analyzes in the time domain, using the wind pressure data obtained from the wind tunnel experiment; a focus is on the deflection of the dome and the resultant member forces, i.e. axial force and bending moment. Based on the results, a discussion is made of the load estimation for the purpose of the wind resistant design of the dome.

WIND-TUNNEL EXPERIMENTS OF WIND PRESSURES ACTING ON THE DOME

The experiments were carried out in a turbulent boundary layer with a power law exponent of $\alpha = 0.24$, which simulated the natural wind over typical urban terrain in Japan. The geometric scaling of the wind tunnel flow is approximately 1/500.

The geometry of the wind tunnel models is schematically illustrated in Fig. 1(a). The span D is 26.7 cm, which corresponds to approximately 130 m in full scale, when assuming that the geometric scaling is 1/500. The surface of the model is smooth. Each model is equipped with 196 pressure taps of 0.5 mm diameter, uniformly distributed on the dome surface. The pressures at all taps were measured simultaneously and sampled in parallel at a rate of 1 kHz on each channel for a period of approximately 33 s. The wind velocity U , at

the level of the roof top was 10 m/s; the corresponding Reynolds number, defined in terms of D and U_r , is approximately 2.2×10^5 . The details of the experimental arrangements and procedures are given in our previous paper (Uematsu et al. (1997)).

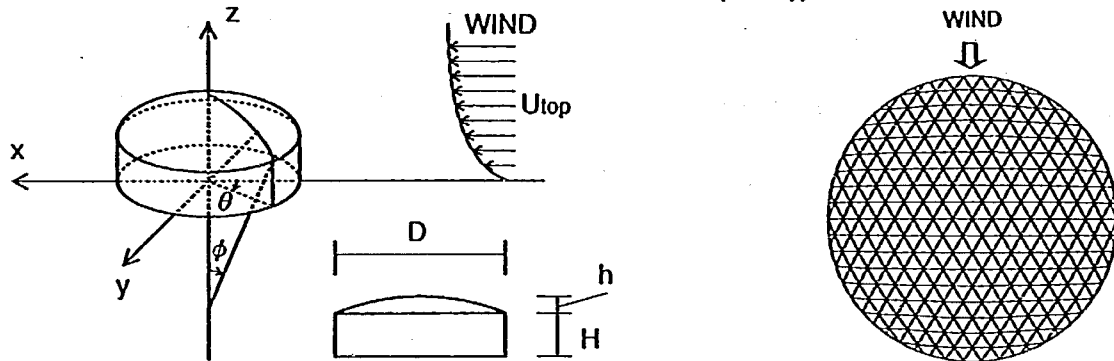


Fig. 1 Geometry and network of the dome (in the case of $N_{div} = 8$).

WIND-INDUCED DYNAMIC BEHAVIOR OF THE DOME

Dome Model

The model for the analysis is a rigidly jointed single-layer latticed dome with a triangular network, constructed of steel pipes (see Fig. 1(b)). The geometry of the dome is similar to that of the above-mentioned pressure model. Table 1 summarizes the dimensions and properties of the models, in which f_1 , N_{div} and q_{cy} represent the first natural frequency, the division number of the network and the buckling load of the dome under external pressure, respectively; q_{cy} is used as a criteria for determining the dimension (outer diameter $\phi \times$ thickness t) of the members. The dead load per unit area of the dome is assumed to be 1.98 kN/m^2 .

Table 1 Dimensions and properties of the dome models analyzed.

No.	D (m)	h/D	N_{div}	ϕ (mm) $\times$ t (mm)	q_{cy} (kPa)	f_1 (Hz)
1	120	0.1	8	508.8 $\times$ 14.0	17.2	1.91
2	120	0.2	8	356.0 $\times$ 14.0	17.6	2.15
3	180	0.1	10	650.0 $\times$ 25.0	18.0	1.44
4	180	0.2	10	500.0 $\times$ 14.0	18.1	1.83

Method of Analysis

The dynamic response of the dome is analyzed in the time domain. The equation of motion for the structural frame is numerically integrated by using the Newmark's β method with $\beta = 1/4$. The time step Δt for the numerical integration is $1/500 \text{ s}$. The wind loads, expressed as the concentrated loads at the interior nodes of the network, are simulated by using the first 25 eigenfunctions of pressure, which are obtained from the proper orthogonal decomposition (POD) technique. The wind velocity is assumed to be $U_r = 50 \text{ m/s}$. In order to examine the dynamic response in each vibration mode, the modal displacements $A_k(t)$ of the 1st to 150th modes are computed.

The dome's response is analyzed for a time duration of 11 min in total. The result for the first 1 min is not used for the statistical analysis of the response because of the non-stationarity. In this paper, a special attention is paid to the deflection and the resultant member stresses at the moment when the response, i.e. the deflection at a node or the stress involved in a member, becomes the maximum.

Results and Discussion

Typical results for the statistics of the modal displacements $A_k(t)$ ($k = 1 - 50$) are shown in Fig. 2. The arrows in the lower figures indicate that the corresponding modes are axisymmetric ones. The results indicate that the response is dominated by several lower axisymmetric and asymmetric modes.

Fig. 3 shows the deflection of the dome ($D = 120$ m, $H/D = 1/4$) at the moment when the maximum deflection w_{max} , the value in the parentheses, is observed; the deflection is normalized by the value of w_{max} . In the case of $h/D = 0.2$, the distribution is approximately axisymmetric; this indicates that the deflection is dominated by the axisymmetric modes. On the other hand, in the low-rise dome case ($h/D = 0.1$), the deflection pattern is asymmetric with respect to the diameter parallel to the wind direction. This feature is related to the fact that the deflection is affected by the asymmetric modes as well as by the axisymmetric modes. The distribution of the axial forces is shown in Fig. 4; the value in the parentheses represents the maximum member stress and the diameter of the circle is proportional to the stress involved in each member. The distribution pattern corresponds well to that of the deflection.

Shown in Fig. 5 are the values of $A_k(t_0)$ at the moment t_0 when the dome's response becomes the maximum ($D = 180$ m, $H/D = 1/4$). The deflection, the axial forces (tension and compression) and the bending moment are considered as the responses. In the case of $h/D = 0.2$, the values of $A_k(t_0)$ for the four responses are almost the same. It is thought that the maximum responses are induced almost simultaneously. In the case of $h/D = 0.1$, on the other hand, the values of $A_k(t_0)$ for the four responses differ significantly.

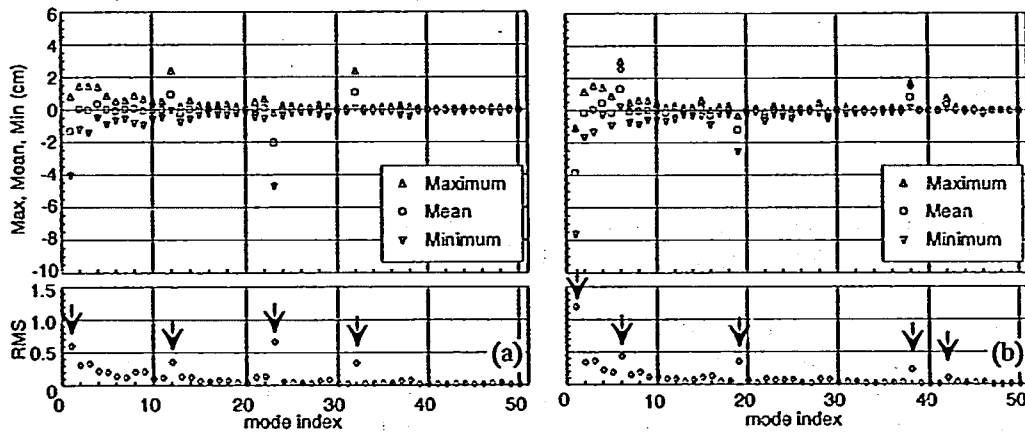


Fig. 2 The statistics of $A_k(t)$: (a) $D = 120$ m, $h/D = 0.1$, $H/D = 1/4$, (b) $D = 180$ m, $h/D = 0.2$, $H/D = 1/4$.

LOAD ESTIMATION BASED ON THE MAXIMUM RESPONSE

Based on the results of the above-mentioned analysis, the design wind load on the roof structure is discussed. The effective load $\{F_e\}$ may be given by the following equation:

$$\{F_e\} = \{\bar{F}\} + \sum_{i=1}^M C_i \operatorname{sgn}(F_i) \{u\}_i \sigma_{F_i} R_i \quad (1)$$

where $\{\bar{F}\}$ = mean wind load; F_i = the i -th modal force; $\{u\}_i$ = the i -th vibration mode; σ_{F_i} = standard deviation of F_i ; and R_i = resonance magnification factor. The critical damping ratio of the structure is assumed to be 0.02 for computing R_i . The selected modes are the

first 6 axisymmetric and 12 asymmetric modes; therefore, $M = 18$. The values of C_i are determined so that the effective load $\{F_e\}$ presents a similar load effect as that by the practical fluctuating wind pressures; the axial force and the bending moment are considered as the load effects. Table 2 summarizes the result, which should be considered as tentative because of the limited range of the models analyzed. As might be expected from the above-mentioned analysis, the contribution of the axisymmetric modes is great for the $h/D = 0.2$ case, while that of the asymmetric modes is relatively great for the low-rise dome case.

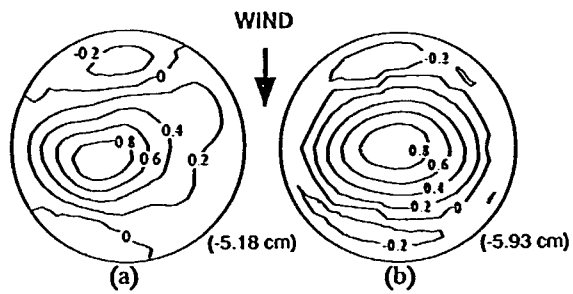


Fig. 3 Distribution of the deflection at the moment when the deflection becomes the maximum ($D = 120$ m, $H/D = 1/4$): (a) $h/D = 0.1$, (b) $h/D = 0.2$.

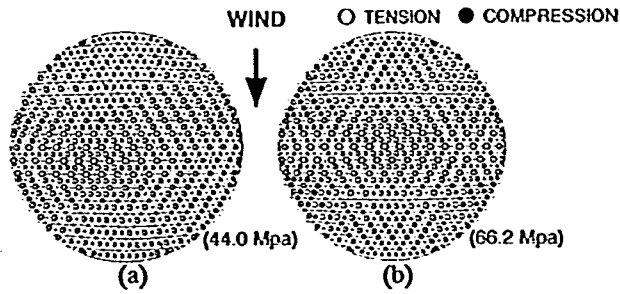


Fig. 4 Distribution of the axial member stress at the moment when the deflection becomes the maximum ($D = 120$ m, $H/D = 1/4$): (a) $h/D = 0.1$, (b) $h/D = 0.2$.

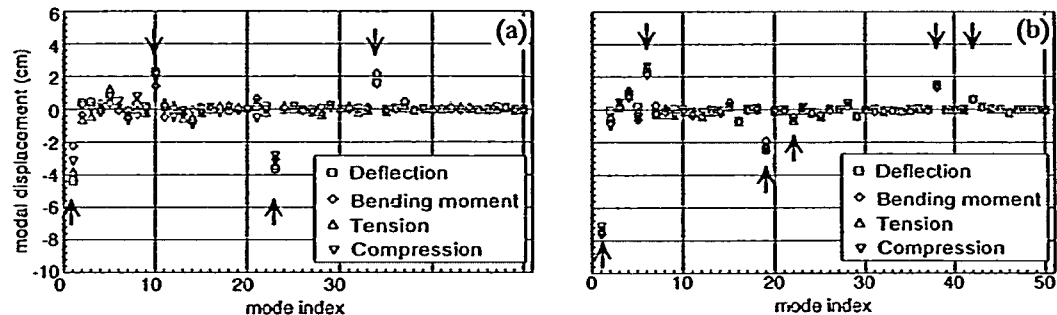


Fig. 5 The values of $A_k(t_0)$ at the moment t_0 when the maximum response is observed ($D = 180$ m, $H/D = 1/4$): (a) $h/D = 0.1$, (b) $h/D = 0.2$.

Table 2 The values of C_i for the effective wind loads

	Symmetric mode						Asymmetric mode	
	1st	2nd	3rd	4th	5th	6th	1st	others
$h/D = 0.10$	2.0	-2.0	2.0	-2.0	-2.0	-2.0	4.0	4.0
$h/D = 0.20$	4.0	-4.0	4.0	-4.0	-4.0	-4.0	4.0	1.0

CONCLUDING REMARKS

The wind-induced dynamic behavior and the resultant load estimation of a single-layer latticed dome with a long span have been investigated, based on a series of dynamic response analyses in the time domain. A simple model of the design wind load is proposed. However, this model is still tentative and should be improved to give a reasonable estimation of the design wind loads for a wide range of the dome's geometry and structural parameters.

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Session 4 Wind Hazards

(Chaired by A. Kareem)

(1) Damage to structures caused by amplified wind speeds in complex topographical terrain.

(Arthur Chiu)

(2) Damages due to strong wind of typhoon 9807 in Kansai area.

(M. Matsumoto, H. Shirato and T. Yagi)

(3) GIS-based wind vulnerability analysis of urban tree populations.

(M. Gaus and D. Liang)

(4) The fujita tornado intensity scale: A reassessment from structural Engineering perspective.

(L. Phan and E. Simiu)

Continued

1941-1942

1. The first year of the war was a very difficult one for the country. The economy was in a state of chaos and the people were suffering from lack of food and shelter.

2. The government was unable to provide the necessary relief and the people were forced to live in a state of constant fear and uncertainty.

3. The war continued to rage and the people were suffering from the effects of the bombing and the loss of their loved ones.

4. The government was finally forced to take action and the people were given some relief from their suffering.

DAMAGE TO STRUCTURES CAUSED BY AMPLIFIED WIND SPEEDS IN COMPLEX TOPOGRAPHICAL TERRAIN

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ABSTRACT

Wind speeds are often amplified as the winds travel over complex topography. Structures located in areas with complex topographic features have been impacted by wind loads that are much higher than normally used for structural design. Although design guidelines have recommended methodologies that attempt to estimate the amplified wind speeds, these recommendations are generally limited to fairly simple topographic profiles. There is a need to obtain information on the effects of complex topography on wind characteristics from wind-tunnel studies and to corroborate these test data with field observations. Such information can then be used to produce wind-hazard zone maps for specifying wind speeds for improved structural design.

INTRODUCTION

Built environments have suffered damaging effects from tropical cyclones in many areas: Caribbean, eastern coastal and Gulf areas of the USA, Hawaii, Guam, South Pacific islands, Philippines, Korea, Japan, Taiwan, eastern coast of China, Hong Kong and the eastern coast of India. Even with improvements in the wind-loads provisions of various building standards and codes, as well as construction procedures, these severe wind storms have continued to take their toll.

Where the terrain changes rapidly from flat land to cliffs and mountains, the wind characteristics can be severely modified. These topographic features can exacerbate the approaching wind speeds and create high local turbulence. During the passage of tropical cyclones in these regions, structures located in areas with complex topographic features, especially those situated at the top of ridges, have often been impacted by wind loads that are much larger than normally used in structural design. The damage caused by Super Typhoon Paka (1997) on Guam and by both Hurricane Iniki (1992) and Hurricane Iwa (1982) on the island of Kauai, Hawaii, are indicative of these destructive forces in areas with complex topography.

Several design standards and guidelines have recommended procedures to estimate the amplified wind speeds, but these recommendations are generally limited to fairly simple topographic profiles. For complex topography, these procedures become useless, and wind-tunnel studies have been used to obtain the answers. The need exists to obtain information on the effects of complex topography on wind characteristics from wind-tunnel studies and to corroborate these test data with field observations. Such information can then be used to produce wind-hazard zone maps for determining appropriate wind speeds for improved structural design.

HURRICANES IWA AND INIKI

The damage caused by Hurricanes Iwa (1982) and Iniki (1992) served as clarion calls to the need for studying topographical effects on wind characteristics. Severe damage to structures and buildings was observed in many areas of complex topography. Hurricane Iwa (NRC 1983) was the first major severe windstorm on record to hit Hawaii and caused more than US \$203 million of property damage. The reported sustained wind speed at the Honolulu International Airport was 49 mph (22 m/s) and a peak gust of 84 mph (38 m/s) from 200 deg. However, on Kauai, which bore the brunt of the storm, the reported sustained wind speed was 73 mph (33 m/s) and a peak gust of 91 mph (41 m/s) from 180 deg. Severe damage was inflicted on buildings and structures, especially in locales of complex topography because of the resulting increased local wind speeds and turbulence. Although Iwa did not hit the island of Oahu directly, the strong winds damaged the transmission lines which then caused a total power blackout on Oahu. The long-span lines traverse over complex topography from ridge to ridge.

During the peak of Iniki, as it crossed the island of Kauai in a northerly direction, the reported sustained wind speeds and peak gusts, respectively, at the Lihue Airport (Chiu et al. 1995) were:

63 mph (28 m/s), 128 mph (57 m/s) from 130 deg at 1502 hrs

96 mph (43 m/s), 114 mph (51 m/s) from 150 deg at 1552 hrs

The hurricane moved northward along the path of the Waimea Canyon. Most of the newly-built residences in the vicinity of the top edge of the canyon walls were destroyed or were severely damaged. This complex topography created a highly turbulent wind environment and the canyon walls also funneled the wind; the combined effects resulted in higher wind speeds.

Reported insured losses of property damaged by Hurricane Iniki exceeded US \$2 billion. The northern side of Kauai, specifically the Princeville area, was severely impacted by Iwa in 1982 and again by Iniki in 1992. Within that area, the coastal topography changes suddenly from sea-level to more than 100 feet (30 m), and this profile caused increased turbulence and speed of the onshore winds over the cliffs. Much of the damage caused by Iniki were similar, and in some cases almost identical, to those caused by Iwa. Because of this repeated pattern of damage, a model study ensued to study the wind flow patterns over escarpments and the distribution of wind loads on low-rise structures (Means 1995). The results showed that wind loads on buildings located on escarpments could be substantially higher than those estimated by building codes, particularly for the middle portions of the roofs.

ESTIMATING WIND SPEED-UP

The ASCE 7-95 Standard (ASCE 1995) has added provisions to estimate wind speed-up over isolated hills and escarpments. Wind pressures can be estimated from Bernoulli's equation:

$$q = 0.5mV^2 \quad (1)$$

in which m is the air mass density and V is the wind speed. To include the variation of wind speed with height, the exposure category in which the structure is located, the importance of the structure, and the mass density of air, the equation is modified:

$$q_t = 0.00256K_zK_{zt}V^2 I \quad (\text{psf}) \quad (2a)$$

$$q_t = 0.613K_zK_{zt}V^2 I \quad (\text{N/m}^2) \quad (2b)$$

in which V is the basic wind speed, I is the importance factor, K_z is the factor for exposure condition and elevation above the standard elevation of 10 m above ground level, and K_{zt} is a factor to account for wind speed-up over hills and escarpments but shall not be less than 1.0. The numerical coefficient of 0.00256 (0.613 for SI units) is to be used unless there are sufficient climatic data available to justify selecting a different value for a specific design application. The provision from ASCE7-95 Standard, Section 6.5.5, for wind speed-up over hills and escarpments states:

"6.5.5 Wind Speed-up over Hills and Escarpments The provisions of this section shall apply to isolated hills or escarpments located in Exposure B, C, or D where the upwind terrain is free of such topographic features for a distance equal to $50H$ or 1 mile, whichever is smaller, as measured from the point at which H is determined. Wind speed-up over isolated hills and escarpments that constitute abrupt changes in the general topography shall be considered for buildings and other structures sited on the upper half of hills and ridges or near the edges of escarpments by using factor K_{zt} :

$$K_{zt} = (1 + K_1K_2K_3)^2 \quad (3)$$

where K_1 , K_2 and K_3 are given in the ASCE Standard. The effect of wind speed-up shall not be required to be considered when $H/L_h < 0.2$, or when $H < 15$ ft (4.5m) for Exposure D, or < 30 ft (9m) for Exposure C, or < 60 ft (18m) for all other exposures."

Detailed procedures for estimating wind speed-up can also be found in the codes of Australia (ASA 1989) and Canada (NRCC 1990).

FIELD MEASUREMENTS

Experience with measuring behavior of wind in Hong Kong addresses the questions associated with predicting flow of air over uneven terrain. Measurements high above the city center have shown that there are several apparent deviations from currently accepted theories as to how the wind should behave when crossing a mountainous area (Jeary 1997). These measurements have shown that the effects of transition from an over-sea profile of horizontal gustiness to that occurring over a city can be seen much closer to the coast than had previously been assumed. Additionally, in Hong Kong and Japan, measurements of turbulence intensity have shown there is a gradual increase as a storm passes a measurement point. The intensity of turbulence is modified by the flow over rough terrain in a way that is currently predictable only in the most generalized form. These effects are symptomatic of the fact that the current description of wind behavior is imperfect. When these deviations from the currently accepted norms are considered together, then, the overall effects on the calculation of the wind force dominates the precision of the estimate. The need is for a full-scale three-dimensional measurement to permit improvement of the precision for understanding the windfield in a complex terrain.

CLOSURE

The magnitude of damage that continues to be inflicted by strong winds on structures located in complex terrain invites the question of the adequacy of the current design methodology. Improvements to the design and construction procedures must be based on measured wind speeds in complex terrain. However, data from field studies are meager.

A concerted coordinated bilateral program, encompassing both model and field measurements, is needed to investigate this phenomenon in detail. The bilateral program could utilize the large wind tunnels available in Japan for model studies of selected areas that contain complex topography, e.g., the Pali area on Oahu or the Princeville area on Kauai. Numerous buildings in the Princeville area were damaged during the passage of Hurricane Iwa (1982) and again by Hurricane Iniki (1992). The joint study could yield insight on the patterns of past damage from these winds and provide the opportunity for corroboration of model test data with field measurements.

The results from this program can be immensely valuable for providing the basis for improving the design guidelines for resistance of structures to these complex winds.

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Damages due to Strong Wind of Typhoon 9807 in Kansai Area

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Abstract

Strong wind disaster due to Typhoon 9807 in September 1998 was investigated through field survey, questionnaire and flow visualization by use of specially manufactured typhoon wind tunnel. Damages to residential houses, other civil engineering structures, forestry, etc. were exceptionally serious for the reported magnitude of this typhoon. The area where strong wind was perceived and its dominant wind direction were analyzed by the answers to the questionnaire as well as some meteorological information. According to the flow visualization, it is pointed out the remarkable influence of the local topography to the air flow especially near the ground surface.

INTRODUCTION

Typhoon 9807, initiated in the South China Sea near Philippine on 17 September 1998, hit the Kansai area of the Japan main island at noon on 22 September, and passed through the inland hilly area toward NE direction, away to the Pacific Ocean in the midnight on the same day. According to the Fire Defense Board, damages were reported from 24 prefectures in Japan including 13 casualties, 2 missing, 59 seriously wounded and 481 slightly wounded. 72 residential houses were completely destroyed, 421 half damaged and 15,553 partially damaged. The damage cost was about 2.3 billion yen. Observed maximum instantaneous wind speed was 59.5m/s (133mile/hr) in a fire department in Nara basin (not coastal area), which caused severe damage of the lattice frame supporting electricity wire of a railway truck, houses, buildings and other structures. However, the magnitude of this typhoon is not so unusually large but can be experienced almost every year in Japan. The effect of local topography on wind speed is discussed based on observed fell down trees, questionnaire and wind tunnel experiment.

METEOROLOGICAL ASPECTS OF TYPHOON 9807

Fig.1 shows typhoon track of T9807. The typhoon passed through the Japan main island from SW to NE direction during only 12 hours. This typhoon track was similar with historical notable typhoons caused serious damage in Osaka area, but was shifted slightly to the east resulting in damage in Nara prefecture. (see Fig.2 (Fujii 1999)) Fig.3 plots time history of the pressure level at the typhoon eye. One of the striking point is that the typhoon indicated its maximum strength just before hitting the main island, whereas maximum influence is likely to

appear enough in advance of hitting in case of usual typhoons. The lowest pressure level was 960hPa, which tells that the strength of this typhoon at this moment was “moderate”.

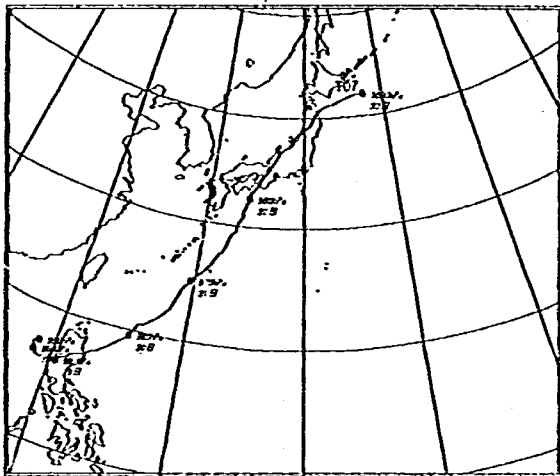


Fig.1 Typhoon track of T9807

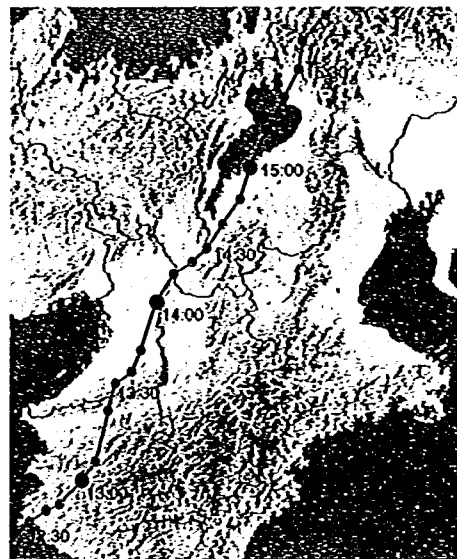


Fig.2 Typhoon track of T9807 in Kansai area (Fujii 1999)

DAMAGE DUE TO TYPHOON 9807

Damage on forestry

Damage on trees was pictured from airplane. Fallen down trees along flight routes observed from airplane are summarized in Fig.4. They are recognized mainly on north side slope along river valleys running east-west direction in general. Most of fallen down direction was N to NE. In detail, however, damage on trees is not distributed continuously but place to place. Some trees fell down in radial direction as shown in Photo 1, although other was changed its fallen down direction sharply by a ridge as shown in Photo 2. The highest damage ratio among cities, towns and villages in Nara prefecture, defined as damaged afforested area divided by total forest area, is over 5%, but actual damage ratio including natural forest must be much higher.

Damage on structures, vehicles

Many roofs of residential houses are partially damaged particularly southern part in Nara basin. A row of lattice frames supporting electricity wire of a railway track was heavily buckled and fallen down by south wind. (see Photo 3) The collapse occurred at 14:40JST

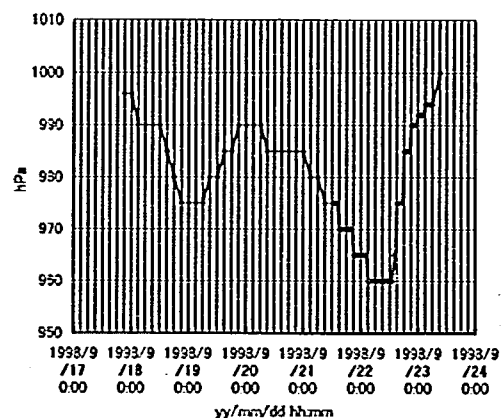


Fig.3 Time history of pressure level at typhoon eye of T9807



Fig.4 Damage on forestry observed during the flights

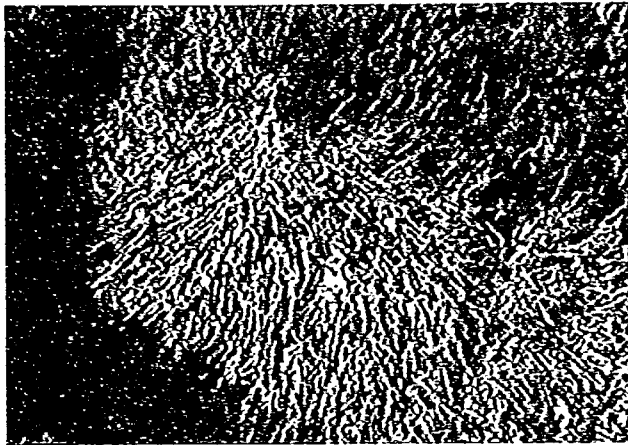


Photo 1 Radially fallen down trees on a hill

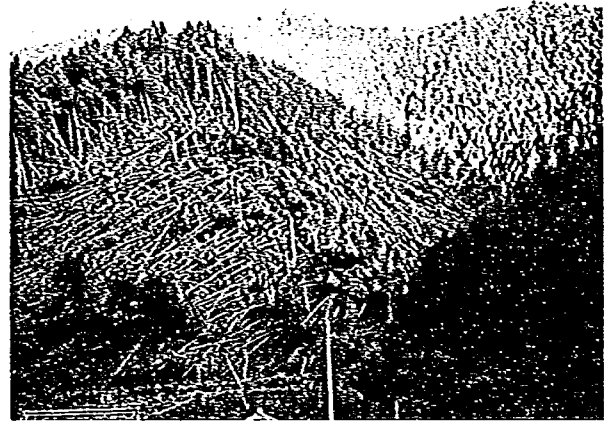


Photo 2 Damaged trees

and the location of the typhoon center was about 50km (31mile) away at this time, although the typhoon approached most at 13:50JST. Damage on historical remains was also serious. Fifty-two national cultural properties suffered damage only in Nara prefecture. There are financial and technical problems how to repair these historical structures as well as how to protect against strong wind. Overturn of vehicles, motor trucks were also reported in several places. Wind speed in case of Photo 4 is evaluated between 44m/s (98mile/hr) to 51m/s (114mile/hr). This evaluated value is comparable with the observed data.

STRONG WIND EVALUATION DURING TYPHOON PASSAGE

The authors sent questionnaire to over 180 public authorities in Nara prefecture in order to capture the distribution of wind direction when the strongest wind speed was perceived during typhoon passage. The same questionnaire was also asked to the habitants during the field survey. Fig.6 summarizes the distribution of strong wind direction in every 15 minutes in northern part of Nara prefecture. The gray colored arrows with wind speed are the observed maximum instantaneous values at fire stations. In this area, the strongest wind

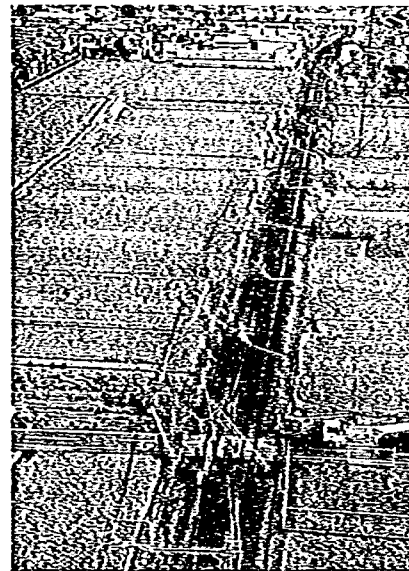


Photo 3 Collapsed lattice frames for electricity wire support (after Yomiuri Newspaper)



Photo 4 Overturned motor truck

speed was mostly perceived from 14:00JST to 15:45JST, although the typhoon eye already passed away from these area in this time. (see Fig.2) It seems that there is dominant air flow from west to east along the valley of Kinokawa river, which curves to north at the southern edge of Nara basin.

Flow visualization was carried out by use of the typhoon wind tunnel and three dimensional topographic model in order to confirm the existence of the air flow mentioned above. The typhoon wind tunnel. It consists of circular table and vertical guide vanes and one small fan at the center of circular ceiling. Fig.7 describes the dominant air flow at 15:00JST. The position of the topographic model was adjusted so that the location of typhoon eye at this time comes to the center on the table. It could be recognized that the air flow near ground surface is especially influenced by the direction of valley.

CONCLUDING REMARKS

Through this study, some remarks can be pointed out as follows:

- 1) The typhoon 9807 maintained its strength just before hitting Japan main island, and ran with high moving velocity. It may cause widely spread severe damage in the inland area.
- 2) There is dominant air flow strongly affected by local topography which could be captured by meteorological data, information by questionnaire, field survey on fell down trees and flow visualization by the typhoon wind tunnel.

Acknowledgement

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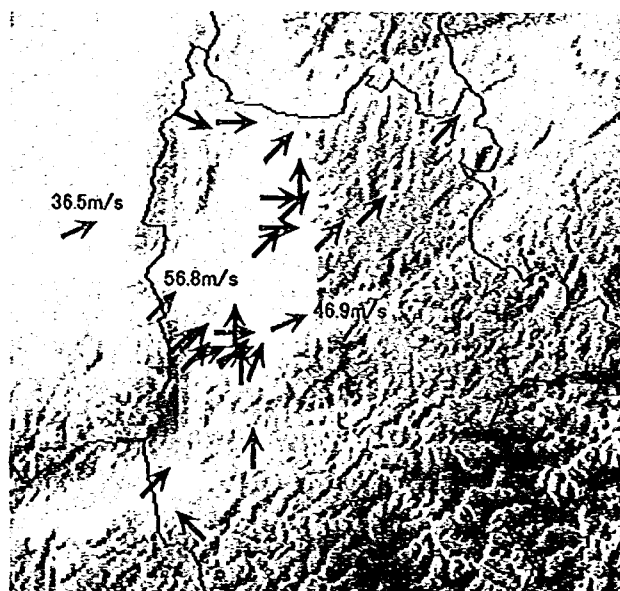


Fig.6 Time and direction of perceived strong wind based on questionnaire (15:00~15:15)

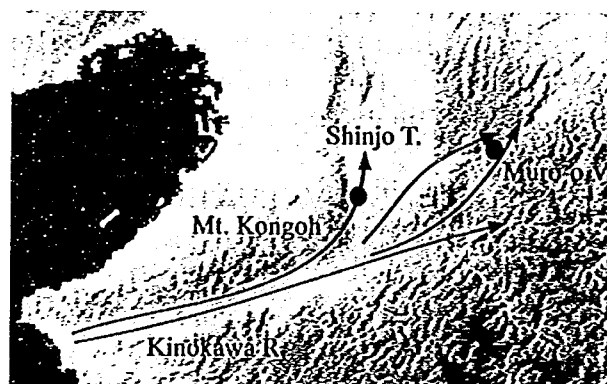


Fig.7 Sketch of dominant air flow at 15:00, Sep.22, 1998

GIS-based Wind Vulnerability Analysis of Urban Tree Populations

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Abstract: Trees are an important part of urban landscapes and can provide both esthetic and environmental benefits. Even though trees are generally appreciated for their beneficial contribution to the urban environment, they can also be a source of hazard or unanticipated expense to communities when there is damage or losses to trees due to natural hazards or other events. Because of the expense of collecting and maintaining databases and in analyzing the resulting information, relatively few communities have had any organized programs to consider tree management or to carry out tree hazard assessments. In the past few years a number of new tools and programming paradigms have become available which provide an opportunity to develop interactive tree management systems. This paper describes the development of a prototype GIS based system for tree management in urban areas with particular emphasis on wind induced tree damage. A major problem is the lack of reliable data for use in developing a tree vulnerability model. This problem will improve in the future as improved data become available.

Introduction

Active management of trees in urban areas has been a desirable goal for many communities for some time but implementation of such procedures has been difficult to achieve because of the expense and difficulties in collecting, manipulating and managing such data. GIS and GPS technologies along with improved statistical analysis programs provide a new opportunity to develop effective interactive urban tree analysis systems. Using modern programming techniques these technologies can be integrated into an interactive analysis system for tree management. The resulting analysis system will have many uses in the better planning and management of tree stocks in urban areas but an additional feature is the capability to carry out risk analysis simulations of hazards posed by tree damage and failure in natural hazard events such as wind, snow, sleet and unusual weather extremes such as drought or extended wet spells. Such a system could be of use in the evaluation of risks and problems associated with trees adjacent to transportation networks.

GIS Procedures for Representation and Analysis of Spatial Data:

A GIS program provides a capability to represent spatial data in a series of layers, which can be superimposed in any desired combinations for analysis or viewing. In this project both the Arc/Info and Arc/View programs have been used. The layers of information are referred to as coverages in Arc/Info or themes in Arc/View. Data can be encoded in either vector or raster formats although for analysis the vector format is preferred. In vector form the data is encoded as points, arcs and polygons. With these relatively simple tools quite complex spatial information can be accurately encoded and displayed. Each spatial feature can be associated with various types of attribute information. For example if a polygon coverage for a particular spatial area has been developed, attribute information may be soil type, soil density, permeability, owner, wind velocity, tax parcel and other similar information. A point

coverage may identify particular trees, their type, height, trunk diameter, crown characteristics and other relevant parameters. If the point coverage is superimposed over the polygon coverage for soil type the relationship between tree type and soil type could be observed.

Another useful feature of the GIS programs is the ability to divide an area into a grid covering the geographic area. Data can then be assigned to the grid squares and various types of analytical procedures performed on the grid data. This procedure is particularly useful where the cost or difficulty of data collection is a factor and statistical approaches become desirable.

GPS – A New Tool to Effectively Build Spatial and Attribute Data Structures

GPS is a technology, which has evolved from defense and space activities and is based on highly accurate time and distance measurements in three-dimensional space. There are two GPS systems currently operational in the world. One is the US Navistar system, which involves 24 satellites, located in 6 orbital planes. The other is the Glonass system developed by the USSR. The basic principle of operation in both cases is the use of triangulation and sophisticated correction procedures to minimize errors in the measurements. Positioning can be based on pseudoranging, which depends upon elapsed time measurements or on carrier phase measurements which measure the number of full wave cycles plus a phase measurement of a partial cycle to obtain distance. Pseudoranging along with differential correction procedures can produce 3-D positional accuracy in a sub-meter range. Carrier phase can produce positional accuracies of as little as 2mm horizontal and 4mm vertical. As may be expected the greater the accuracy of the measurement, the greater the cost of equipment and time required for obtaining data. For tree surveys it would only require sub-meter accuracy and small GPS units are available at relatively modest cost which can provide instantaneous position measurements with real-time differential correction using local stations broadcasting correction signals. The units also provide a user interface in which attribute data can be entered directly. Tree height information is also needed for vulnerability analysis but this information can easily be obtained with a total station instrument and the data included in the attribute database. Sophisticated computation routines are programmed into the GPS systems so that output can be produced in any desired coordinate system such as UTM or State Plane. If needed Geoid corrections could be downloaded for a particular area and used to find orthometric elevations. Positional and attribute information can be directly downloaded from the GPS system into GIS compatible databases without intermediate processing. Thus with the GPS units, a one or two man crew could quickly develop an urban tree database using statistical procedures on grid squares or from total population measurements. Ideally this would be done both before and after an extreme wind event.

Characteristics of Trees:

The characterization of trees can involve a very high level of complication with respect to damage or loss vulnerability in windstorms. Just a few of the factors which could be considered would be tree type, age, height, root structure, crown development, trunk diameter, general state-of-health and seasonal effect on foliage. If the study being carried out is for general urban tree management many of these parameters may have to be collected. In the initial development of a wind hazard vulnerability study the importance of various parameters is evaluated and a reduced set parameters identified for analysis.

Wind Loading on Trees:

Ideally it would be desirable to characterize the load-response-failure properties of each type of tree along with its' parameters. This information could then be applied to each representative group of the tree population in an urban area or even to each individual tree if this level of detail was utilized. There is not a large amount of data on the wind drag on trees and the resulting mechanical performance or possible failure of the trees. Such information has been and could be developed in a number of ways. One would be to carry out field tests on actual trees using pull tests on calm days to get mechanical force-deflection behavior and then to make subsequent deflection measurements on windy days to calibrate wind drag force. Extrapolation to failure conditions would have to be calibrated with actual tree failures. Linearity problems in extrapolating between different stages of tree development would have to be examined. It would be difficult to make accurate measurements of wind-drag characteristics of model trees in a wind tunnel because of the extreme complexity of actual trees and their supporting conditions. Because of the difficulties and expense involved in making actual tree measurements, a regression-based vulnerability model was developed using a small amount of loss data, which was available from Charleston, South Carolina and was used for the prototype study development.

A Regression-based Model for Tree Vulnerability:

Regression analysis is a statistical technique for investigating and modeling the relationship between variables. It is widely applied in almost every field, such as engineering, economics, management, biology and social sciences. It proves to be one of the most effective statistical techniques.

In regression analysis the first thing which should be done is to select the form of the model. For tree damage prediction, a model was built based on multiple linear regression. This model, based on the presumed theoretical considerations is as follows:

$$y = \beta_0 + \beta_1x_1 + \beta_2x_2 + \beta_3x_3 + \beta_4x_4 + \dots + \beta_kx_k + \varepsilon$$

The regression equation contains two types of variables: *independent* variables X_k 's and *dependent* variables Y . A nonlinear regression model could also be constructed however even if some or all X s behave in a nonlinear fashion, a linear regression model will provide much useful information. This makes the linear approach useful for the development of a vulnerability model. Until improved quality and quantity of data become available it does not appear that the nonlinear model will give superior results over the linear model.

For initial model development the following independent variables were selected:

TYPCUR _i	i=1 to 10	type of tree
HEIGHT		height of tree
AGE		age of tree
SOIL _i	i=1 to 5	type of soil
ROUGH _i	i=1 to 3	roughness of terrain
GUST		peak gust speed of wind

There is one response (dependent) variable:

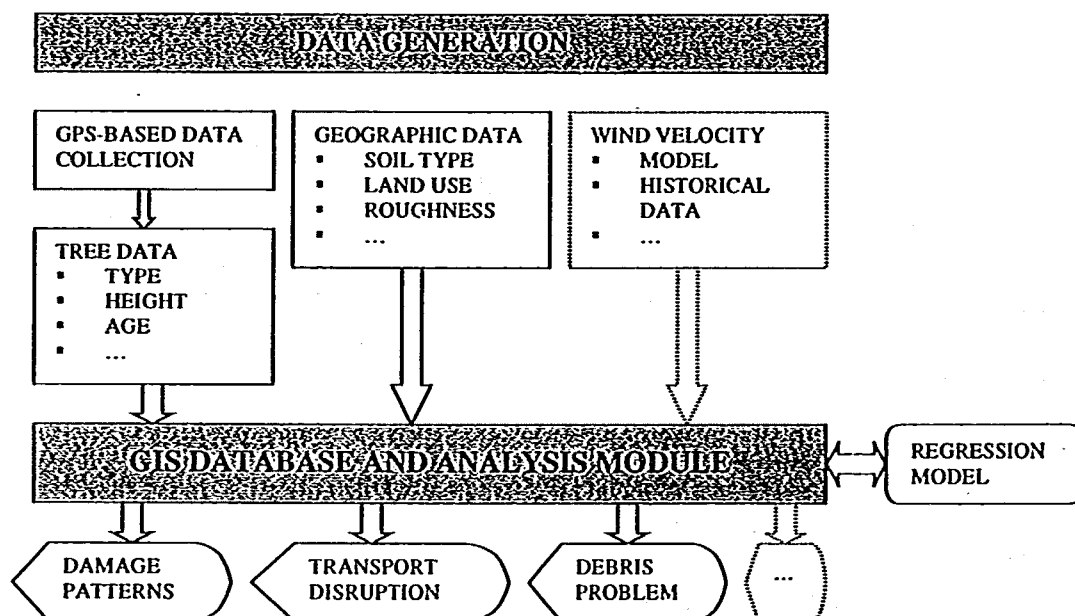
DAM	damage of tree
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Some of the variables may be deleted and new variables added in the future based on the results of model verification.

This model was developed and calibrated using information reported for hurricane Hugo that struck Charleston, South Carolina in 1989. Using the actual damage information it was found that the values of Soil3 and Soil6 were small and that these two variables do not have a major effect and can be eliminated from the model. From the SPSS output it was found that Coefficient of Determination (R^2) is 0.881. This means that 88.1% of the variation in Damage can be explained by the variation in the independent variables, which were selected. At this point this model performs quite well but will be improved in the future with additional data.

The GIS-based Vulnerability Analysis System

The flow diagram below shows the primary modules of the prototype analysis system: Base maps could utilize TIGER-type data for general analysis or higher resolution base maps if they have been developed for a study area. The initial program utilizes a uniform wind distribution over the study area but it would be a simple matter to include a dynamic polygon-based distribution if the information is available. After the program is loaded with basic data, a user can select various scenarios from menus such as wind velocity, season, etc. The program will then evaluate the desired parameter such as level of damage and distribution of this damage. Results can be displayed as graphic coverages or in tabular report format. This information will be useful for planning for the impacts of extreme wind events including potential levels of damage, the disruption of transportation routes damage to utility systems and the magnitude of the tree debris disposal, which must be dealt with after the disastrous event. The simulation system will also be of use in evaluating potential strategies of removing the most potentially troublesome existing trees and planning future urban tree populations to provide the most satisfactory performance.



The Fujita Tornado Intensity Scale: A Reassessment from Structural Engineering Perspective

by

Long T. Phan¹ and Emil Simiu²

ABSTRACT

On May 27, 1997 the town of Jarrell in central Texas, and on May 30, 1998 the town of Spencer in South Dakota were hit by two violent tornadoes. We evaluated the structural damage caused by the Jarrell tornado and concluded that the worst damage can be explained by wind speeds corresponding to an F3 rating on the Fujita tornado intensity scale (wind speeds of 71 m/s to 92 m/s). An F4 (93 m/s to 116 m/s) rating, or the F5 (117 m/s to 142 m/s) rating officially issued by the National Weather Service (NWS), need not be assumed to explain that damage. We also present assessments of damage in the Spencer tornado, and suggest there is an inconsistency between the NWS ratings of the Jarrell and Spencer tornadoes. We ascribe the NWS ratings to the failure of the Fujita scale to account explicitly for the dependence of wind speeds causing specified types of damage upon two structural engineering factors: (1) the basic design wind speed at the geographical location of interest, and (2) quality of construction, defined as degree of conformity to applicable standards requirements. We address the need for a stronger involvement of the structural engineering profession in nationwide efforts to develop an improved tornado classification scale and a more realistic tornado windspeed database.

KEYWORDS: Building technology; Fujita tornado intensity scale; meteorology; standards; structural engineering; wind engineering; tornadoes.

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1. INTRODUCTION

On May 27, 1997 a violent tornado swept through a housing area on the outskirts of Jarrell, Texas (population 410) in Williamson County, approximately 60 km north of Austin, Texas, destroying about 40 single family residences and killing 27 people. A year later, on May 30, 1998 a violent tornado struck the town of Spencer, South Dakota, a small farm community approximately 72 km west of Sioux Falls, leaving 6 dead, more than 150 injured, and nearly 90 percent of a total 195 structures in the six-by-seven blocks community destroyed. Immediately following the passages of these tornadoes, structural engineers of the National Institute of Standards and Technology (NIST), in collaboration with meteorologists of the National Weather Service (NWS), conducted post-storm inspections to document structural damage and to assess tornado intensities. (Both NIST and NWS are agencies of the U.S. Department of Commerce.) This paper (1) describes the most severe structural damage observed for the Jarrell and Spencer tornadoes, which served as the basis for the NWS official rating of F5 and F4, respectively (see Appendix I); and (2) assesses the tornado wind speeds in Jarrell based on engineering considerations. From this assessment it was concluded that the damage caused by the Jarrell tornado may be explained by wind speeds corresponding to an F3 rating. An F4 rating, or the F5 ratings officially issued by the NWS, need not be assumed to explain that damage. The apparent misclassification of the Jarrell tornado can be ascribed to two shortcomings inherent in the current Fujita tornado intensity scale. The first shortcoming is that the damage descriptors for tornado categories F3 to F5 include ambiguous terms referring to construction quality, such as "well constructed houses," "structures with weak foundation," and "strong frame houses." These

descriptions are open to subjective interpretations and can lead to inconsistent field applications, especially by non-engineers who normally cannot be expected to have the requisite technical knowledge needed for ascertaining whether or not houses destroyed by tornadoes were well-constructed or strong. Evidence of subjective application of the Fujita tornado intensity scale is further illustrated by comparing the damage caused the Spencer tornado and that of the Jarrell tornado. While similar damage to similar structures was observed in Spencer and Jarrell, the Spencer tornado was rated as an F4 tornado instead of F5, as was the case for the Jarrell tornado. The second shortcoming is that the Fujita tornado intensity scale does not reflect the dependence of the tornado wind speeds causing various types of damage upon the design wind speed specified for the zone of interest. For example, a tornado wind speed that tears down the roofs of well constructed houses may be assumed to be larger by a factor of roughly 1.6 for zones in which the design wind speed is 63 m/s than for zones where the design wind speed is 40 m/s; that is, damage that in a 63 m/s zone could be attributed to an F5 tornado could be explained in a 40 m/s zone by F3 tornado winds.

2. THE JARRELL TORNADO

2.1 Post-storm Survey

Shortly before 3:45 PM CDT on May 27, 1997, the Jarrell tornado first struck areas of Bell County causing minor damage to trees and the roofs of a few residences. The tornado then crossed the Bell/Williamson county line into Williamson County where the town of Jarrell was located. The locations where the tornado crossed county roads were clearly marked by strips of scoured asphalt road surface. The top layer of the asphalt pavement was about 20 mm thick. The scouring of asphalt road surface is ascribed to differences between the atmospheric pressure in pockets of air trapped underneath the asphalt and the lower atmospheric pressure at the tornado center. The tornado path length was estimated to be 12.2 km and the maximum path width was measured to be approximately 1.2 km. Witnesses estimated that

the Jarrell tornado was moving at a slow speed of between 2 m/s and 4 m/s.

The NIST-NWS post-storm survey team conducted ground surveys, on May 29 and June 1, 1997, and two aerial surveys: one on May 30, 1997 using a fixed wing aircraft flying between 760 m and 910 m, and the second on May 31, 1997 using a helicopter flying between 150 m and 300 m. A commercial Global Positioning System (GPS) was used to obtain the coordinates defining the tornado path.

2.2 Main Damage Observations

The first structural damage in Jarrell occurred at the corner of County Road 305 and County Road 307, where a culvert plant collapsed in place (Figure 1). The plant was a light steel frame construction with non-reinforced masonry and steel tube columns supporting steel pipe, gable roof trusses. The steel tube columns were cast into a concrete mat foundation on grade. The roofing, supported by wood purlins, consisted of corrugated sheet metal. An identical plant located approximately 15 m away from the collapsed culvert plant, and a mobile home located about 150 m north-northwest of the plant sustained only very minor damage. As the tornado continued its track south-southwestward, a barn located just southwest of the culvert plant and just north of the Double Creek Estates subdivision in Jarrell was destroyed. The tornado then moved through the Double Creek Estates subdivision and the surrounding areas, where the destructive path widened to its maximum of 1.2 km and the tornado became most deadly.

Figure 2 shows a portion of what remained of Double Creek Estates subdivision after the tornado passage. The structures in the subdivision consisted mainly of single family residences built over the last 15 years or so, of the typical slab-on-grade construction type. Owing to the presence of a limestone bedrock most houses had no basements or underground shelters. The houses, which consisted mainly of wood frame construction, were completely swept off the concrete foundations. Inspections of the concrete

slab-on-grade foundations performed during the ground surveys revealed that, in many cases, even the sill plates that connect the wood frames to the concrete slab-on-grade foundations were blown away (see Figure 3). There was evidence that the sill plates had been connected to the foundations by explosive-driven nails. Figure 4 shows a nail that remained in the foundation but pulled through the sill plate. Figure 5 shows nails, spaced 0.9 m apart, that pulled out of the foundation. The Council of American Building Officials (CABO): One and Two Family Dwelling Code and other nationally recognized model codes require that sill plates of exterior walls for wood-frame construction be anchored to the foundation by anchor bolts not more than 1.83 m apart, but no evidence of anchor bolts was found by the team. It should be noted that Williamson County *has not adopted a building code*, even though Jarrell was also hit by a tornado on May 17, 1989.

3. THE SPENCER TORNADO

3.1 Post-storm Survey

The Spencer tornado struck at 8:38 pm CDT on May 30, 1998 and moved through the town of Spencer at a speed of approximately 15-20 m/s. The tornado left a ground track about 34 km long and close to 1.5 km wide at its broadest. An aerial survey using fixed wing aircraft flying at between 760 m to 910 m was conducted on the following day, and was followed by a ground survey one day later.

Based on surveys of the most severe structural damage, the NWS assigned it an F4 rating. Reportedly, a Doppler-On-Wheels (DOW) radar, operated by the School of Meteorology of the University of Oklahoma as part of the Radar Observations of Tornadoes and Thunderstorms Experiments (ROTATE-98) program, was able to record wind speeds of this tornado and indicated wind speeds of up to 110 m/s. These measurements, if validated, would be consistent with the wind speed range associated with an F4 tornado (93-116 m/s).

3.2 Main Damage Observations

Figure 6 shows an overview of the town of Spencer after the tornado passage. Within the town of Spencer, the observed damage ranged from total disintegration of structures in the direct path of the tornado, to light damage to envelopes of structures in the outlying areas. The structures affected varied in age and structural systems. Completely destroyed structures included one and two-story wood-frame, heavy timber, and engineered metal frame constructions. Among these were wood-frame constructions that were either poorly connected or totally unconnected (i.e., relying on gravity alone) to the concrete or brick foundations. However, structures that were totally destroyed also included some (post office, fire department, gas station, antique and other stores) that were properly anchored to the concrete slab-on-grade foundations by 12.7 mm (½-in) anchor bolts with proper spacing as required by CABO (1995) and other nationally recognized model codes (see Figures 7 (a) and (b)). Many heavy trucks, some reportedly fully loaded, used in transporting grains to and from the town's grain processing and storage warehouses, were lifted and carried more than 30 m away by the wind. Trucks and houses exhibited widespread evidence of missile impacts from broken wooden power poles, tree branches, and other debris.

4. TORNADO INTENSITY ESTIMATES

The Jarrell and Spencer tornadoes offered an opportunity to assess subjective tornado intensity estimates based on Fujita scale damage descriptions. Our assessments are based on structural engineering considerations.

The basic peak gust wind speed specified in the ASCE 7-95 Standard (1995) for the design of buildings in central Texas is 40 m/s. Building designs for central Texas before 1995 should be roughly compatible with this basic speed. Design and construction practices of buildings such as those destroyed by the Jarrell tornado are normally based on experience and judgment rather than formal engineering calculations. When they are sound, such practices are consistent with the

requirement that buildings should be capable of withstanding wind speeds higher than the basic design wind speed, with a safety margin with respect to wind loading of at most 1.5 to 2. Assume that in areas with a 40 m/s basic design wind speed buildings would collapse under winds loads twice as large as the loads induced by the 40 m/s speed. It follows from the proportionality of the wind loads to the square of the wind speeds that those buildings would be expected to collapse under wind speeds in excess of $40 \times (2)^{1/2} = 56.6$ m/s. The expectation would be even stronger that wind speeds corresponding to wind loads 3 times as large as those induced by the 40 m/s speed (i.e., 69 m/s wind speeds) would leave no buildings standing, especially if the construction is mediocre or poor, as appears to have been the case for buildings destroyed by the Jarrell tornado.

Speeds assigned to F3 tornadoes in the Fujita classification are about 72 m/s to 94 m/s. For a building to resist such speeds its safety margin with respect to wind loads would on average have to be about 3.1 to 5.5. There is no reason to believe that any of the buildings destroyed by the Jarrell tornado were that strong. We note that 90 m/s wind speeds, which in the Fujita classification are associated with F3 tornadoes, would likely destroy residential homes not only in a 40 m/s basic design wind speed zone, but in hurricane-prone areas with 63 m/s basic design wind speeds as well.

In fact, based on evidence that code compliance with respect to the connections between the upper wood frame and the concrete foundation was better in Spencer than in Jarrell, one may conclude that buildings that were completely destroyed were stronger in Spencer than in Jarrell. Nevertheless, the NWS tornado ratings were F5 and F4 for the Jarrell and Spencer tornadoes, respectively. In our opinion this is a clear indication of possible inconsistencies inherent in subjective tornado speed assessments by non-engineers.

5. CONCLUSIONS

Our conclusions are:

1. The strongest damage caused by the Jarrell tornado can be explained by wind speeds corresponding to an F3 rating (i.e., 71 m/s to 92 m/s).
2. The Fujita tornado intensity scale has two major shortcomings that can lead to misclassification of tornadoes by non-engineers:
 - (a) It does not reflect the dependence of the tornado wind speeds causing various types of damage upon the design wind speed specified for the zone of interest.
 - (b) The damage descriptions include ambiguous descriptions of structural quality such as "well-constructed houses" and "strong frame houses." Non-engineers normally cannot be expected to have the requisite technical knowledge needed for ascertaining whether or not houses destroyed by tornadoes were well-constructed or strong.

These conclusions are of significance insofar as:

1. Tornado misclassification corrupts the database used to develop design criteria for structures whose performance must be unaffected by strong tornadoes.
2. Ascribing failures to unrealistically high wind speeds when the actual speeds are in fact lower discourages the application and enforcement of standards, such as ASCE 7, that are capable of reducing loss of life and property caused by most tornadoes.
3. A stronger contribution of the structural engineering profession to efforts aimed at assessing tornado wind speeds could help improve the knowledge needed to protect the public from tornado-induced losses.

For these reasons, we recommend the followings:

1. Engineers, meteorologists, disaster relief workers, and representatives of standards organizations, regulatory bodies and the insurance industry should work together to develop a tornado intensity classification scale wherein damage descriptions make specific reference to basic design wind speeds and to quality of construction as defined by degree of conformity to standards requirements. As a basis for discussion, it is suggested that wind speed ranges included in tornado intensity classifications be affected by numerical factors related to the basic design wind speeds and to quality of construction. Factors related to quality of construction could be specified in accordance with matrices of the type considered for development by insurance companies, wherein conformity to various standards requirements is estimated both qualitatively and quantitatively.

2. ASCE, in collaboration with other interested parties, should consider organizing and training local ASCE chapter volunteer engineering teams that could join in expeditiously with local NOAA and other specialized personnel, and promptly record from the strength of their professional knowledge significant evidence on tornado intensities. This recommendation is based on experience which shows that evidence of tornado effects is often removed from the site less than a day after a tornado occurrence, that is, *before* survey teams can reach the site.

3. The documentation for the approximately 150 tornadoes (about 0.5% of the total number of recorded tornadoes in the U.S.) that were classified as F5 should be revisited in an attempt to reassess their classification. Such reassessment would involve a relatively small effort and would, even if only partly successful, allow some

updating of the database from which probability distributions of tornado wind speeds have been estimated so far. This recommendation follows a proposal by K. Mehta of Texas Tech University.

4. Continuing efforts to develop procedures for obtaining direct, scientific tornado wind speed measurements should be encouraged. On the one hand such measurements would add valuable data to the database, even though the effect of such addition to the more than 25,000 data currently available would be small: many years of data would have to be collected for that effect to be significant. On the other hand, scientific measurements can help to assess estimates of tornado wind speeds based on observations of damage. This would in turn help to effect corrections to the existing database by allowing the use of Bayesian or other updating techniques.

6. REFERENCES

ASCE 7-95, Minimum Design Loads for Buildings and Other Structures, ASCE Standard, American Society of Civil Engineers, New York, New York.

The Council of American Building Officials, *CABO One and Two Family Dwelling Code*, 1995 edition, Falls Church, Virginia.

Long T. Phan and Emil Simiu, *The Fujita Tornado Intensity Scale: A Critique Based on Observations of the Jarrell Tornado of May 27, 1997*, NIST Technical Note 1426, National Institute of Standards and Technology, July, 1998.

Long T. Phan and Emil Simiu, *Tornado Aftermath: Questioning the Tools*, ASCE Civil Engineering Magazine, December, 1998.

APPENDIX I. FUJITA TORNADO INTENSITY SCALE

Category-Definition-Effects

F0 Gale tornado (Approximate wind speeds 18-32 m/s (40-72 mph)): Light damage. Some damage to chimneys; break branches off trees; push over shallow rooted trees; damage sign boards.

F1 Moderate tornado (Approximate wind speeds 33-50 m/s (73-112 mph)): Moderate damage. Lower limit is the beginning of hurricane wind speed; peel surface off roofs; mobile homes pushed off foundations or overturned; moving autos pushed off the roads.

F2 Significant tornado (Approximate wind speeds 51-70 m/s (113-157 mph)): Considerable damage. Roofs torn off frame houses; mobile homes demolished; boxcars pushed over; large trees snapped or uprooted; light-object missiles generated.

F3 Severe tornado (Approximate wind speeds 71-92 m/s (158-206 mph)): Severe damage. Roofs and some walls torn off well-constructed houses; trains overturned; most trees in forests uprooted; heavy cars lifted off ground and thrown.

F4 Devastating tornado (Approximate wind speeds 93-116 m/s (207-260 mph)): Devastating damage. Well constructed houses leveled; structures with weak foundation blown off some distance; cars thrown and large missiles generated.

F5 Incredible tornado (Approximate wind speeds 117-142 m/s (261-318 mph)): Incredible damage. Strong frame houses lifted off foundations and carried considerable distance to disintegrate; automobile-sized missiles fly through the air in excess of 100 yards; trees debarked; incredible phenomena will occur.

(From National Post-storm Data Acquisition Plan, Federal Coordinator for Meteorological Services and Supporting Research, National Oceanic and Atmospheric Administration.)

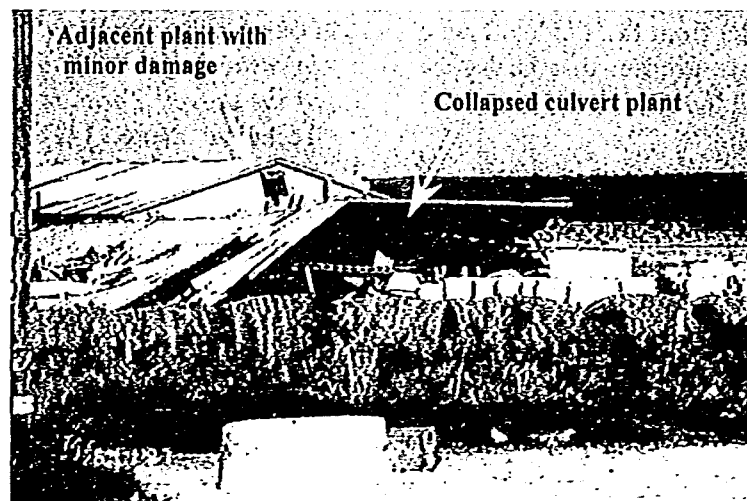


Figure 1. Ground view of collapsed culvert plant.



Figure 2. Aerial view of portion of the Double Creek Estates subdivision after tornado passage

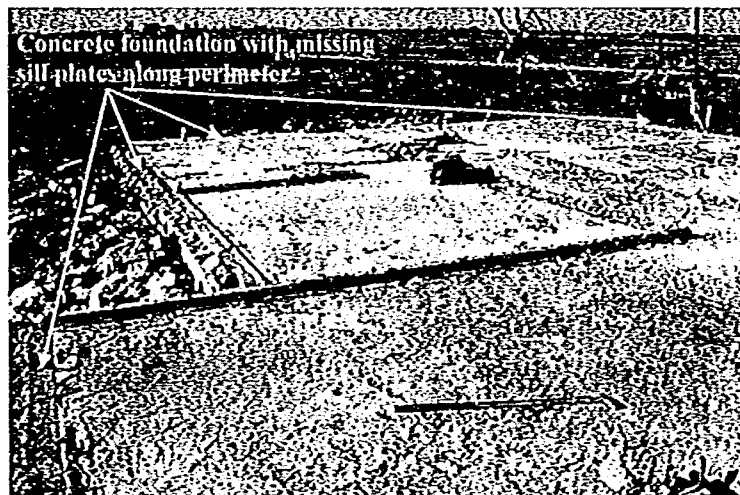


Figure 3. Close-up view of a typical slab-on-grade foundation of houses in Jarrell. Only one sill plate remains connected to the concrete foundation.



Figure 4. Evidence of nail that was used for connecting sill plate to concrete foundation.

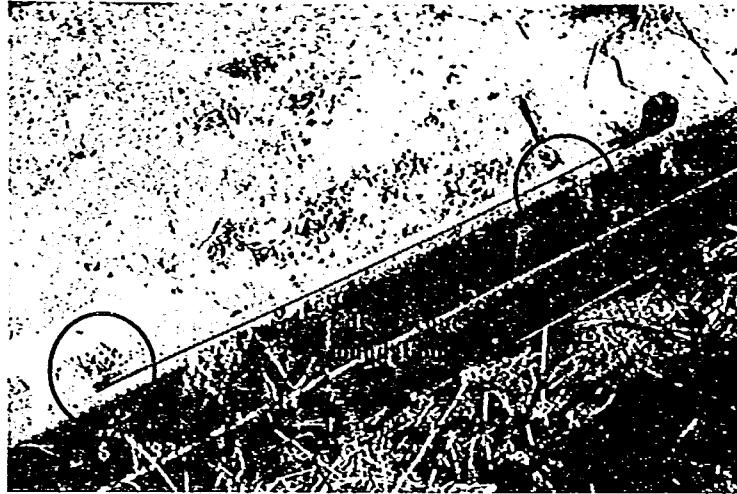


Figure 5. Evidence of pullout of nails used to fasten sill plates to concrete foundation.

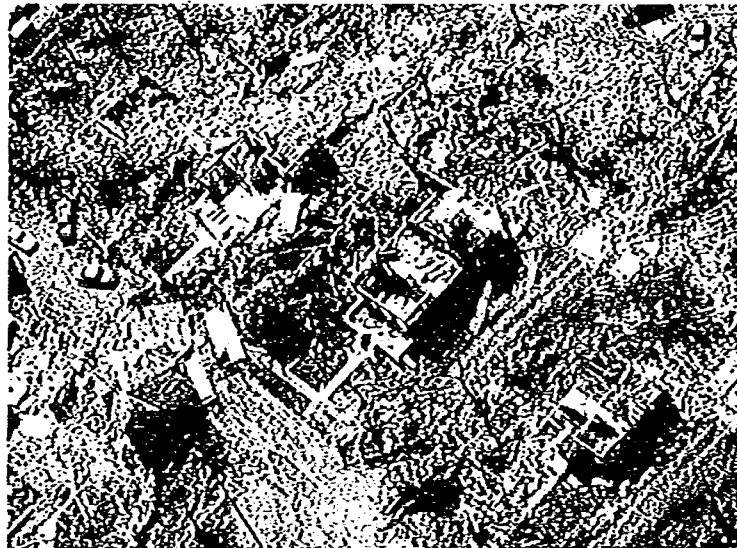
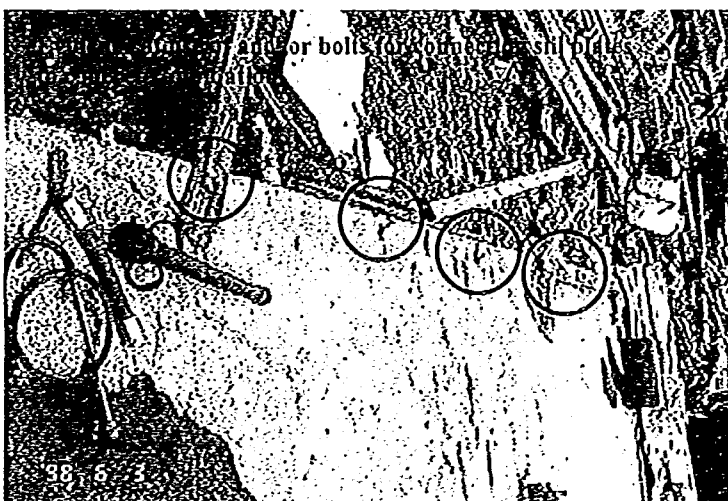
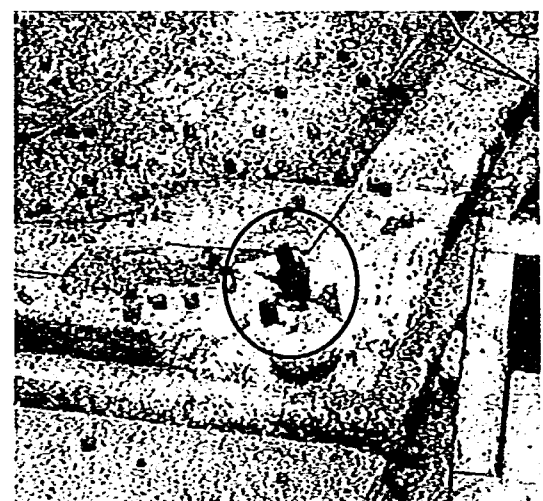


Figure 6. Aerial view of portion of the town of Spencer, South Dakota, after tornado passage.



(a)



(b)

Figures 7 (a) and (b). Evidence of anchor bolts used to connect upper structure to concrete slab-on-grade foundation in Spencer, South Dakota.

Appendix A

WORKSHOP AND STUDY-TOUR PROGRAM

23 May 1999 (Sunday)

U.S. members arrive in Japan (Narita airport).

Travel to Tsukuba by train or bus

Lodging at Hotel Shinonome in Tsukuba.

488-1 Onozaki, Tsukuba-shi, Ibaraki-ken, 305-0034

tel. 0298-56-2211

24 May 1999 (Monday)

0900-1200 Visit Meteorological Research Institute (MRI), Building Research Institute (BRI), and Public Works Research Institute (PWRI).
A drive-through visit of Tsukuba University and the Tsukuba Science City.

1200-1300 Lunch.

1300-1400 Opening ceremony at PWRI.

Welcome and opening remarks

Y. Inoue, Director-General, Public Works Research Institute

J. Golden, US-side Chair of Task Committee E

H. Sato, Japan-side Co-Chair of Task Committee E

Introductions (all participants)

Photograph

1400-1700 Technical presentations.

1400-1510

Session 1: Wind Characteristics (Chaired by Dr. M. Yagi)

J. Golden, NOAA: The wind engineering of extreme windstorms: How well do we know the windspeed distributions?

Y. Tanaka, Meteorological Research Institute: 10km and under 1km scale wind speed fluctuation observed in typhoon area.

Mark Powell, Hurricane Research Division/NOAA, T. Reinhold and J. Golden: Understanding wind speeds in hurricanes: Implications for meteorological and engineering applications.

N. Furuya, R. Toriumi, Honshu-Shikoku Bridge Authority: A study on spatial correlation of natural wind.

Kevin K. Knupp, University of Alabama at Huntsville: Wind Profiler Observations of Hurricane GEORGES, 1998 at landfall.

Elizabeth A. Ritchie and Russell Elsberry, Naval Postgraduate School: A modeling study of the development of mesoscale asymmetries in tropical cyclone

structure during landfall.

1540-1700

Session 2: Wind Flow, Pressure and Loading (Chaired by Dr. A.Chiu)

H.Okada, BRI: Comparison of wind pressure measurements on a tower-like structure between full-scale observation and wind tunnel test.

Partha P. Sarkar, K.C. Mehta, and D.A. Smith, Texas Tech University: Roof corner wind flow and pressures.

Y. Tamura, Tokyo Institute of Polytechnics: Extreme load distributions on buildings with a rectangular plan.

Gregory Chiu, City University of Hong Kong: Performance-based objective design and natural disaster risk management.

T. Tamura, Tokyo Institute of Technology: Feasibility of CFD technique for prediction of wind flows and wind loading.

B. Bienkiewicz, Colorado State University: Towards refined modeling of wind effects through collaborative research.

1800-2000

Reception, hosted by Director-General, PWRI.

Lodging at Kenshu Kaikan (MOC Guest House) in Tsukuba.

2 Nishizawa, Tsukuba-shi, Ibaraki-ken, 300-2624

tel. 0298-64-2844

25 May 1999 (Tuesday)

900-1200 Technical presentations.

900-1030

Session 3: Wind-induced Response(Chaired by Dr. Y.Tamura)

Nicholas P. Jones and Robert Scanlan, Johns Hopkins University: Long-span bridge response to wind: Recent developments.

K. Kimura, U. of Tokyo: Wind-induced response characteristics of bridges in the wake of a hill.

Ahsan Kareem, Fred L. Haan, and Albin A. Szewczyk, Notre Dame University: Effect of turbulence on the space-time characteristics of the self-excited forces on a rectangular cross-section.

H. Sato, PWRI and N. Furuya, HSBA: Cooperative study between Public Works Research Institute and Honshu-Shikoku Bridge Authority on wind resistant design of super long-span bridges.

Dorothy Reed, T. Mabuchi, H. Sato, G. Turkiyyah and H. Yeh, Univ. of Washington: Development of computational models for bridge deck motion.

H. Kikitsu, BRI: Experimental study on reducing aerodynamic response of tall buildings with open passage design.

Y Uematsu, Tohoku U.: Wind-induced dynamic behavior of a single-layer latticed

dome.

1100-1200

Session 4: Wind Hazards (Chaired by Dr. A.Kareem)

Arthur Chiu, U. Hawaii: Damage to structures caused by amplified wind speeds in complex topographical terrain.

M. Matsumoto, H. Shirato, Kyoto U.: Damages due to strong wind of typhoon 9807 in Kansai area.

Michael Gaus and Daan Liang, State University of New York at Buffalo: GIS-based wind vulnerability analysis of urban tree populations.

Long Phan, NIST: The Fujita tornado intensity scale: A reassessment from structural Engineering perspective.

1200-1300 Lunch.

1300-1700 Discussion on collaborative research.

1800-2000 Dinner.

Lodging at Kenshu Kaikan (MOC Guest House) in Tsukuba.

2 Nishizawa, Tsukuba-shi, Ibaraki-ken, 300-2624

tel. 0298-64-2844

26 May 1999 (Wednesday)

0900-1100 Discussion and adoption of Resolutions.

1100-1200 Closing ceremony.

Closing remarks

J. Golden, US-side Chair of Task Committee E

H. Okada, Japan-side Co-Chair of Task Committee E

1200-1300 Lunch.

1300- Travel from Tsukuba to technical visit site by train

Lodging and dinner at Green Hill Hotel in Onomichi.

9-1 Higashi-gosho-cho, Onomichi-shi, Hiroshima-ken,

722-0036 tel. 0848-24-0100

27 May 1999 (Thursday)

0900-1700 Technical site visits:

Western route of Honshu-Shikoku bridge to see various types of long-span bridges (Hiroshima-ken to Ehime-ken).

1800-2000 Dinner.

Lodging at Green Hill Hotel in Onomichi.

9-1 Higashi-gosho-cho, Onomichi-shi, Hiroshima-ken,

722-0036 tel. 0848-24-0100

28 May 1999 (Friday)

**Travel from Onomichi to Kansai or Narita airport by train.
U.S. members depart from Kansai or Narita airport.**

Appendix B

**UJNR/Panel on Wind and Seismic Effects
Task Committee E Workshop
May 24-26, 1999
International Conference Room
Public Works Research Institute
Tsukuba, Ibaraki, JAPAN**

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**UJNR/Panel on Wind and Seismic Effects
Task Committee E Workshop
May 24-26, 1999
International Conference Room
Public Works Research Institute
Tsukuba, Ibaraki, JAPAN**

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Appendix C

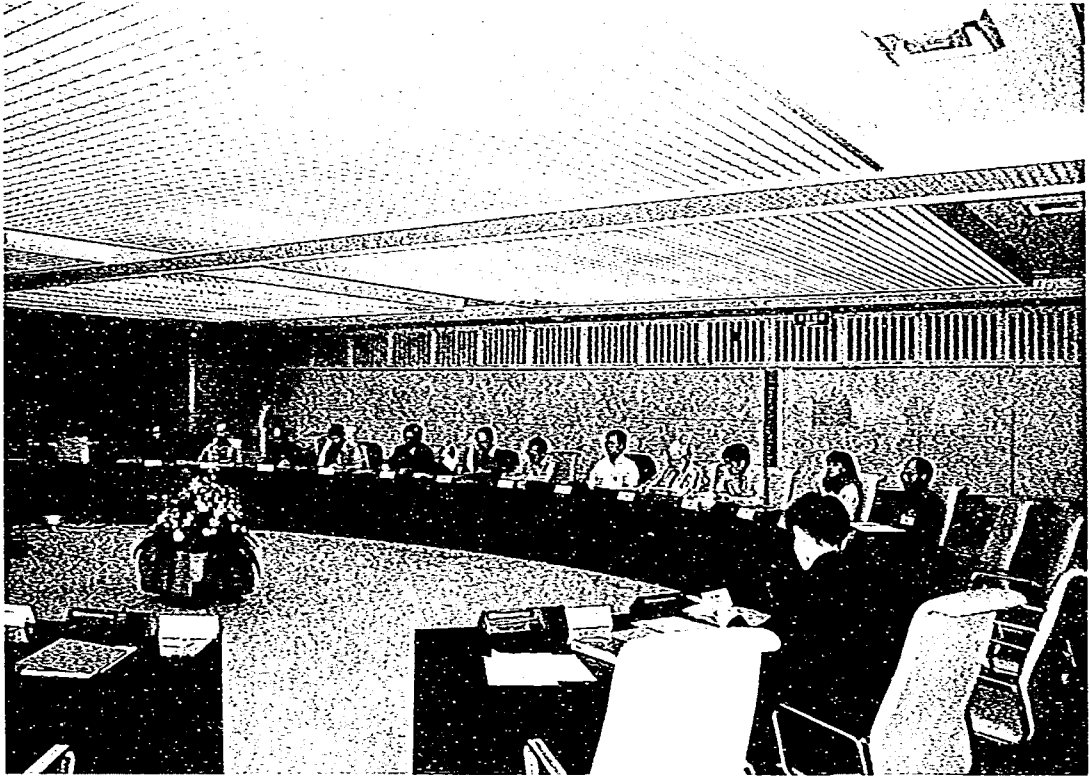


Photo 1 U.S. side members

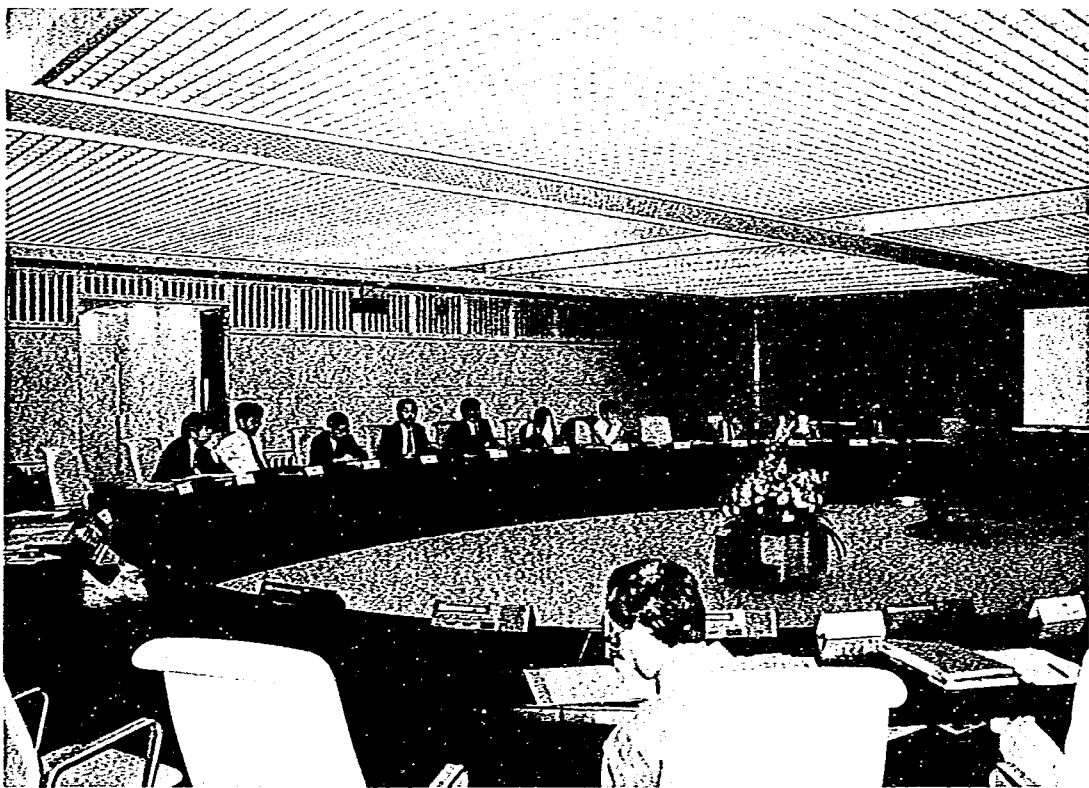


Photo 2 Japan side members

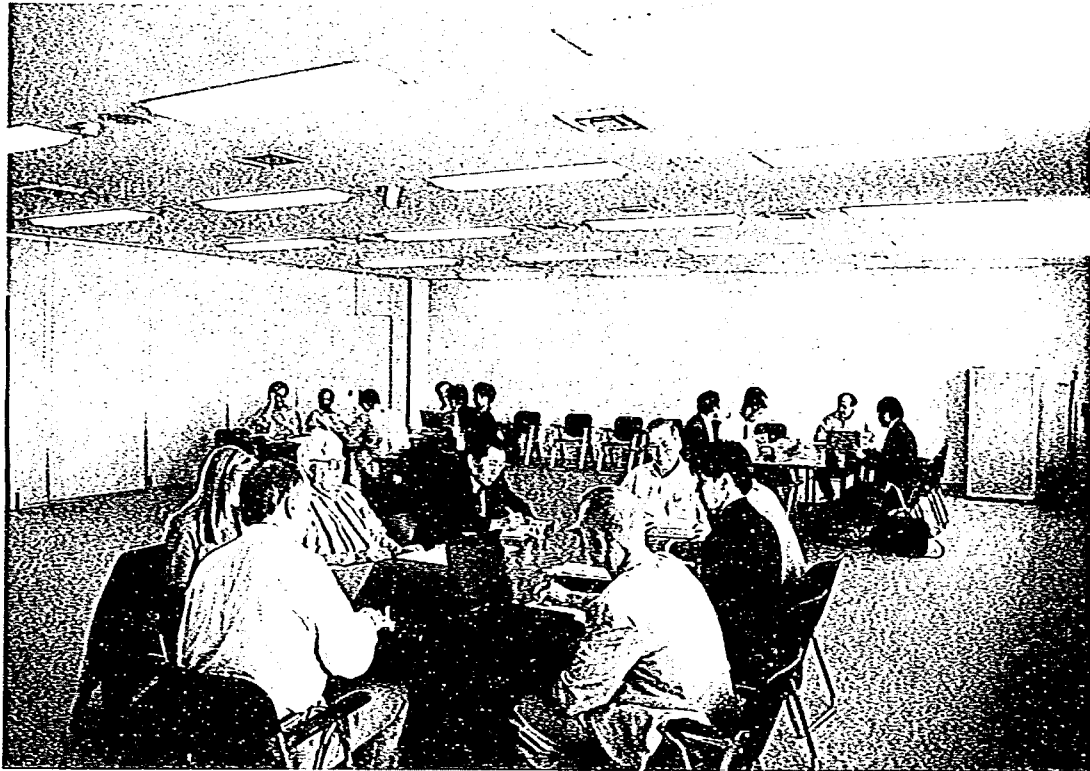


Photo 3 Discussion on collaborative research



Photo 4 At the wind tunnel of Public Works Research Institute

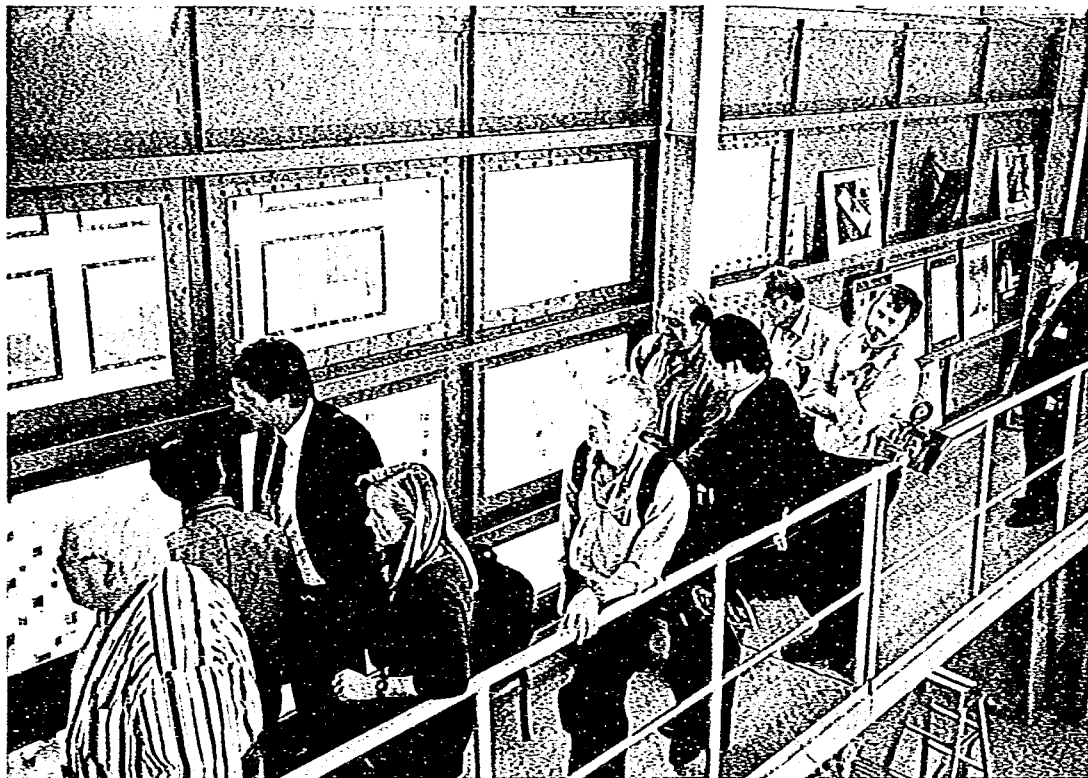


Photo 5 At the wind tunnel of Building Research Institute

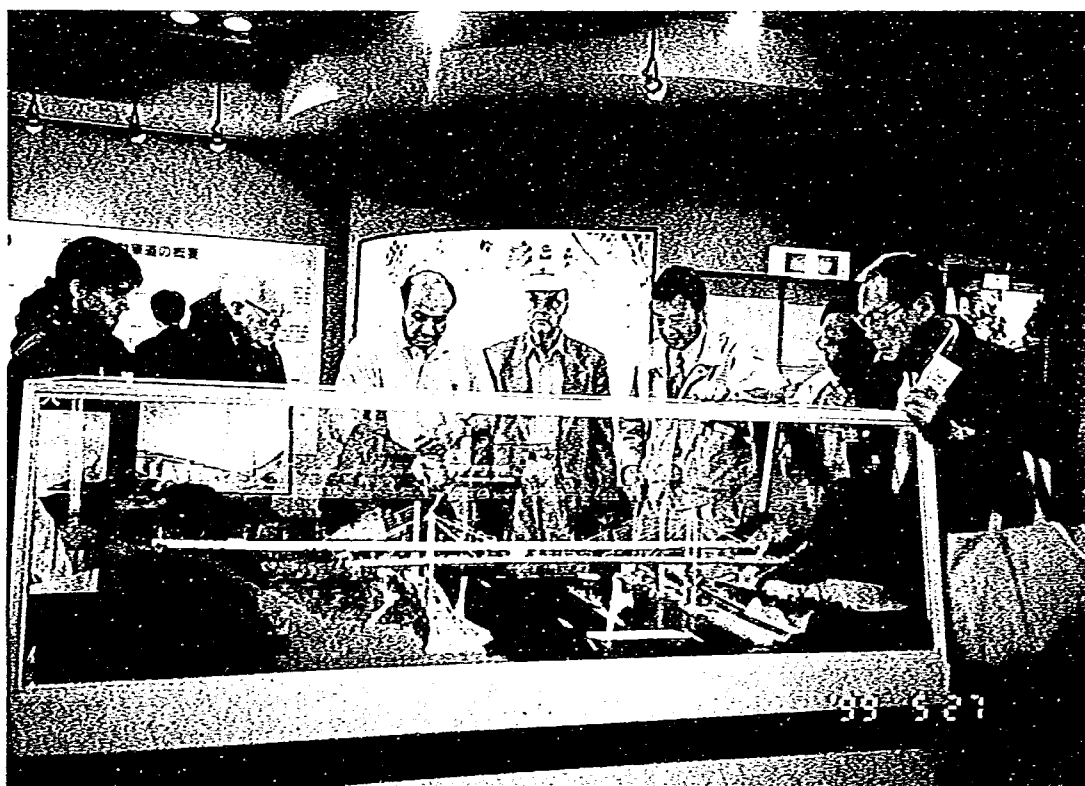


Photo 6 At the 3rd Operation Bureau, Honsyu Shikoku Bridge Authority



Photo 7 On the Kirousan observation platform

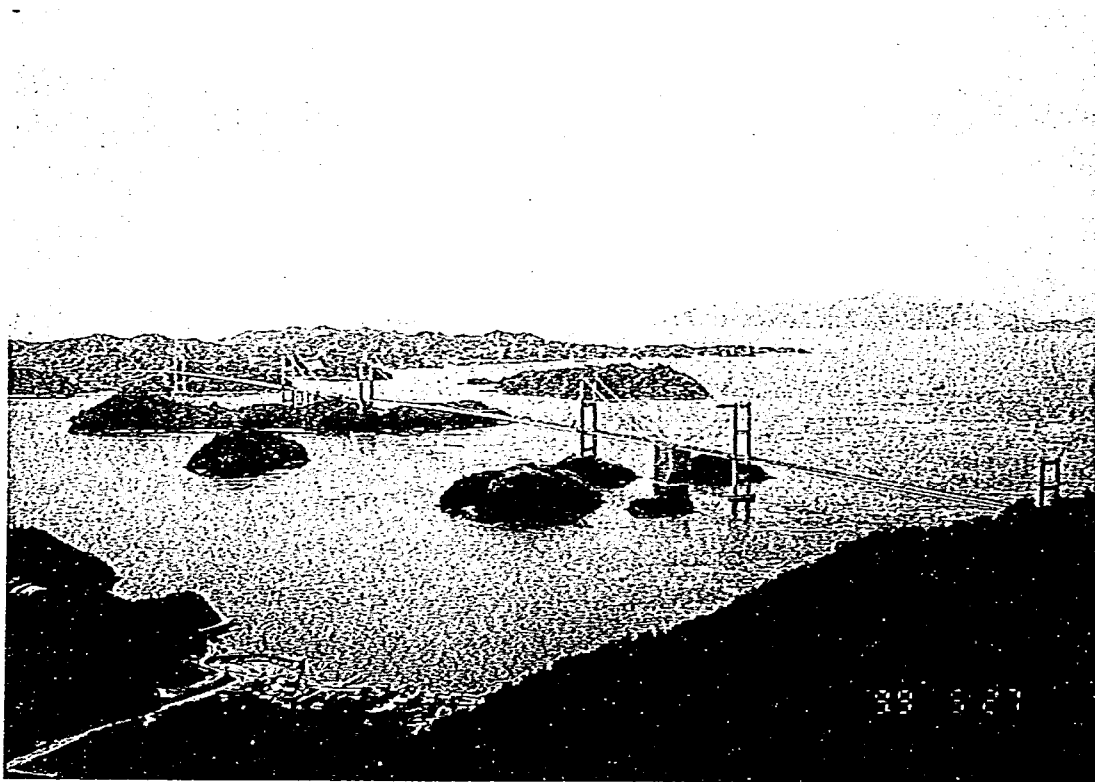


Photo 8 Panorama of the Kurushima-Kaikyo Bridges from Onomichi side

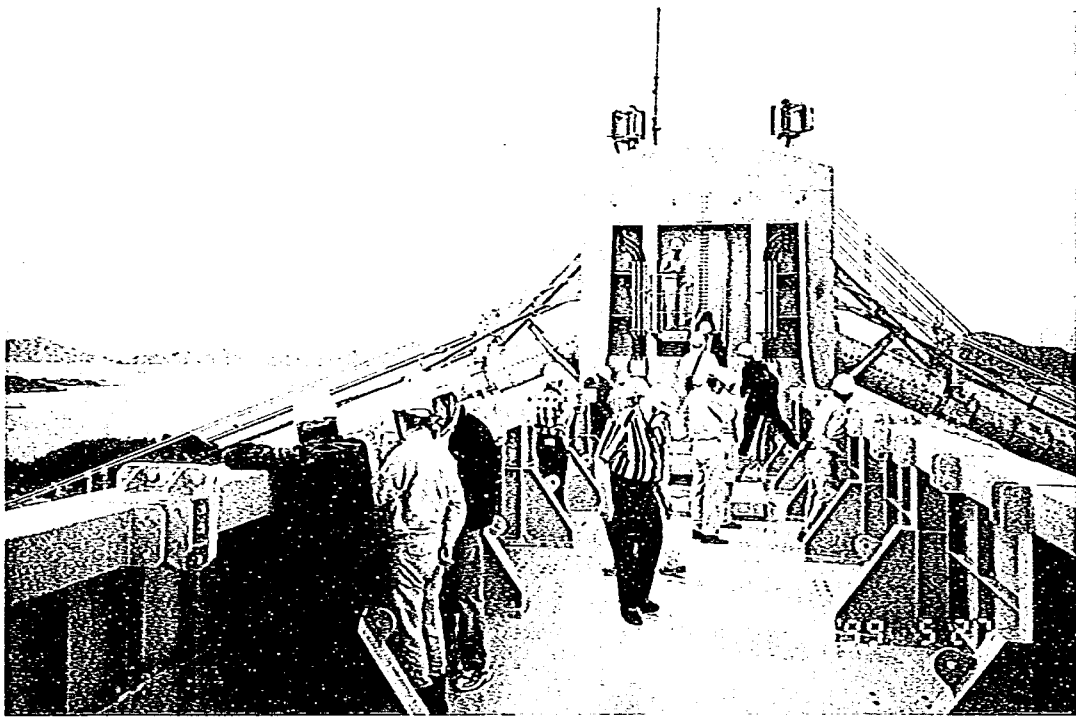


Photo 9 On the top of the main tower of Kurushima Bridge

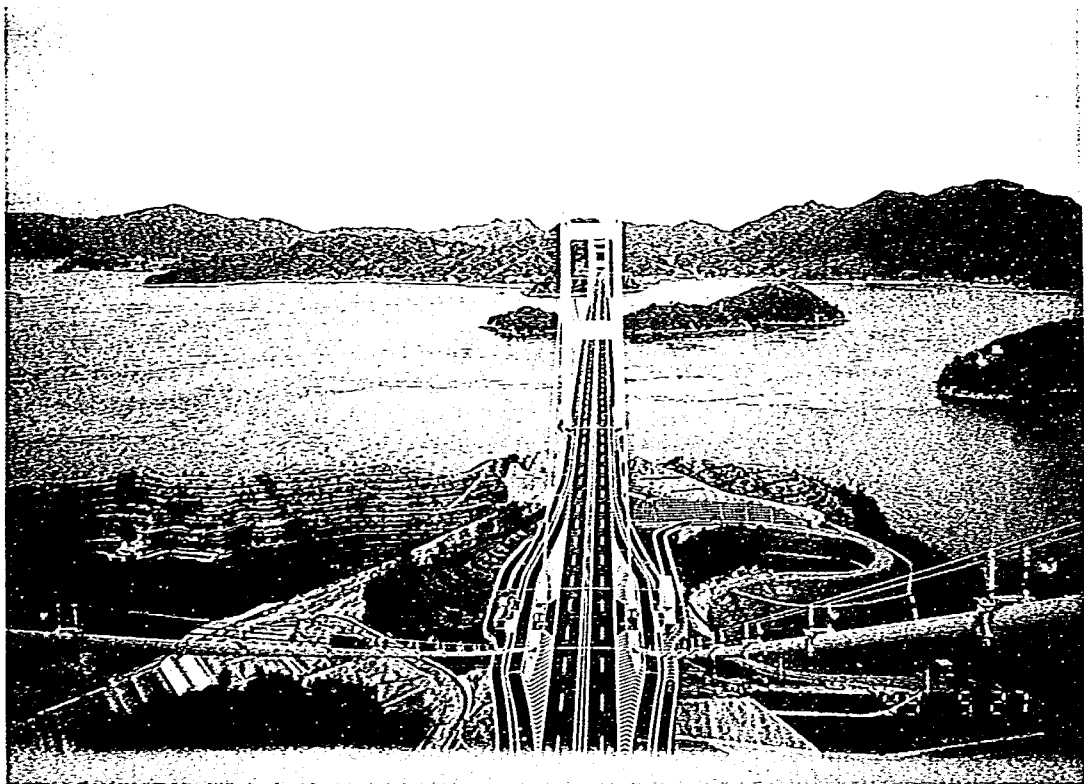


Photo 10 Kurushima-Kaikyo Bridges from the top of main tower